Conceptual solutions to minimise the effects of cobbles on the sand-bypassing system at the Port of Ngqura.

by
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Declaration of Authenticity

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Abstract

The Port of Ngqura constructed two breakwaters to create safe anchorage for vessels at berth. These coastal structures obstructed the natural movement of sand alongshore. A fixed sand-bypassing system was installed to transport the obstructed sand from the up-drift to the down-drift side of the Port. The system failed to reach the design rate; leading to major sand accretion against the western breakwater.

The main problem was the presence of coarse material causing an obstruction at the jet pumps that prevents further bypassing of the material, located in the sandtrap. The system was designed to handle particles with a diameter of 150 mm, but it was the presence of particles larger than 150 mm that particularly created a problem. Two alternative fixed sand-bypassing systems, the Nerang river and Tweed river bypassing schemes, were investigated to determine if similar problems arose and if lessons could be learned for application at the Port of Ngqura. To find a solution to this challenge, it was necessary to determine where these particles originated from in the first part of the study.

It was found that the coarse material originated from mainly three sources: temporary construction works required for the construction of the bypassing system, the rock revetment behind the sand-bypassing jetty, as well as from natural sources. While the origin and properties of the first two sources were known, further investigation was required to determine the source of the natural coarse material. The objectives of this part of the study were to gain a better understanding of the principles of sediment transport and particle motion as well as to investigate the origin, volume and properties of the natural source. The Swartkops river was found to be the main source of coarse material for the western section of Algoa Bay, with an estimated mean annual volume of 150 m^3.

Using the findings gained in this first part of the study, five viable solutions were conceived to prevent the obstruction at the jet pump intakes. All five solutions are considered viable solutions to the current problem at hand, but some were deemed more viable than others. For some of the conceptual solutions to function to their full potential, modifications were also required at the revetment and sandtrap. The sandtrap modifications included the removal of all the coarse material currently located in the sandtrap. For the revetment modifications, the armour layer at a certain section should be reconstructed and units that do not meet the requirements should be replaced.

The study concludes that each proposed solution together with the required sandtrap and revetment modifications can serve as a potential solution to achieve the design bypassing rate of the fixed sand-bypassing system at the Port of Ngqura. The conceptual solution that proved the most promising, however, is the pile-and-mesh structure due to the relatively small impact that this proposed solution would have on the surrounding coastline, the low maintenance required, and high capacity of the structure.

Opsomming

Die Hawe van Ngqura het ’n sandverbyvoering-sisteem geïnstalleer om die sand wat deur die hawe se breekwaters geblokkeer word, van die opwaartse kant na die afwaartse kant te vervoer. Ná die eerste paar jaar het die sisteem nie die teiken volume bereik nie, wat gelei het tot ’n sandopbouing teen die westerse breekwater.

Daar is verskeie redes hoekom die sisteem nie die teiken volumes bereik het nie, maar een probleem het herhaaldelik voorgekom. Die sisteem was oorspronklik ontwerp om sediment kleiner as 150 mm te verwyder, maar daar is steeds sediment met kleiner diameters in die vanggat teenwoordig. Die probleem was die teenwoordigheid van growwe sediment om die inlaat van die jet pomp, wat veroorsaak het dat die pompe nie verder kon funksioneer nie.

Deur deeglike ondersoek is dit bepaal dat die growwe sediment afkomstig is van die tydlike strukture wat benodig was vir die konstruksie van die sisteem, die rotsbeskutting, asook natuurlike bronne. Vir die eerste twee bronne genoem was die oorsprong en eienskappe bekend, maar oor die laasgenoemde bron was daar meer inligting nodig.

Die doelwit van die studie was om te bepaal wat die natuurlike bron was, die volume wat dit verskaf, asook die metode wat dit vervoer word. Daar is gevind dat die Swartkopsrivier die hoofbron in die gedeelte van Algoa Baai is, en dat daar ’n jaarlikse volume van 150 m^3 gelewer word.

Die literatuurstudie is hoofsaaklik gebaseer op die manier waarop growwe sediment in die kusstelsel beweeg en dit dan te vergelyk met kleiner sediment soos sand. Die diepte tot waar die growwe sediment in die kusstelsel beweeg, is ook bepaal. Met die bogenoemde inligting was dit moontlik om konseptuele oplossings te vind om die blokkasie by die inlaat te vermei.

Vyf konseptuele oplossings is ontwerp om te verhoed dat die pompe in die toekoms geblokkeer word. Elke oplossing is gebaseer op ’n spesifieke beginsel vir growwe sedimentvervoer wat gelei het na die algemene idee. Die ligging, ontwerp, installasie, kapasiteit en bekommernisse van elke oplossing word deeglik verduidelik in die literatuurstudie.

Verskeie van die konseptuele oplossings het veranderinge benodig aan die rotsbeskutting en vanggat voor die oplossing behoorlik kon funksioneer. By die vanggat was dit nodig om al die growwe materiaal wat tans teenwoordig is, behoorlik te verwyder. Vir die rotsbeskutting is die hoof- en onderlaag herontwerp om te verhoed dat die eenhede in die lae kan beweeg.

Dit kan dus geargumenteer word dat elke voorgestelde oplossing kan dien as ’n oplossing vir die probleem om ten einde die teiken verbyvoering te bereik by die Hawe van Ngqura.

Acknowledgements

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List of acronyms and abbreviations

Abbreviation	Designation or connotation
CD	Chart datum
EIA	Environmental impact assessment
HAT	Highest astronomical tide
LAT	Lowest astronomical tide
ML	Mean level
MSL	Mean sea level
PRDW	Prestedge Retief Dresner Wijnberg
SLR	Sea level rise
TNPA	Transnet National Port Authority

Chapter 1: Overview

1.1 Background

Today, one of the most important considerations in the development of any new coastal project is to predict the potential environmental impact of such a project in order to provide opportunities for both mitigating possible negative impacts and enhancing the positive effect of such a project.

With the development of a new port in South Africa, one of the basic elements in the design is the use of breakwaters. Breakwaters are constructed with the purpose of forming an artificial harbour protected from the effect of waves in order to provide safe berthing areas for vessels. These coastal structures are extended seawards to create safe conditions beyond the harbour structure too. Unfortunately, such extensions can also cause an interruption in the natural movement of sand along shorelines. After a period of time, sand tends to build up at the up-drift (in the upward direction of the net longshore transport) side of the port; causing severe erosion on the down-drift (in the downward direction of the net longshore transport) shoreline because of the resultant lack of sand supply.

If left unattended, sand will eventually bypass the up-drift breakwater naturally; causing sand to move into the entrance channel of the port. The prevalence of sand in the entrance channel will not only affect the safety of vessels making use of the port, but may also pose serious consequences for the coastline on the down-drift side, including loss of land, which poses a threat to human settlements; coastal recreation; harbours; as well as various other problems.

A potential solution for the interrupted sand transport includes the use of artificial sand-bypassing methods, which transfer the trapped sand from the up-drift side to the down-drift side. Such bypassing can usually be effected using one of two methods. First, bypassing can be done by dredging or excavating the sand at the up-drift side by using dredgers or heavy machinery and transporting it to the down-drift side by a land or sea-based method (Coastal and Hydraulics Laboratory, 2008). The second method involves transporting a mixture of sand and water from the up-drift to the down-drift side with the help of pumping equipment and piping. Both these methods for artificial sand-bypassing have a number of variations due to the specific conditions and unique requirements of each location.

The case that forms the subject of this study, the Port of Ngqura, installed a fixed sand-bypassing system making use of jet pumps to create a sand-water mixture. During the construction of the system, however, several problems arose; causing the system to open five (5) years after the port construction commenced. One of the main reasons for this delay was the existence of coarse material in the sandtrap; originating not only from natural sources but also from temporary rock protection works. These coarse materials obstructed the intake of the jet pumps, preventing the system from reaching target bypassing rates. With the bypassing rate not matching the natural longshore transport rate, accretion occurred on the up-drift side, which was estimated at 1 million m^3.

If this problem is not resolved in the near future, the impact of the Port of Nqgura on the surrounding environment will likely deteriorate further. Not only will the continued accretion therefore violate the terms of agreement for constructing the Port of Ngqura, but it may also harm the surrounding shoreline of Algoa Bay.

1.2. Objectives and aims

The objectives of this study include the following:

1. Understanding the entire fixed sand-bypassing process at the Port of Ngqura, as well as to point out the reason(s) for not achieving the design bypassing rate.
2. Determining the origin of coarse material found in the sandtrap.
3. Understanding the principles of coastal sediment transport and utilising the difference between the method of transport for fine and coarse grained material.
4. Producing conceptual solutions to prevent coarse material from obstructing the intakes of the sand-bypassing system.

The aim of this study is to help to enable the sand-bypassing system to reach the design rate in order to match the natural longshore transport rate prior to the construction of the Port of Ngqura. This will thus ultimately help to mitigate the impact of the Port on the Algoa Bay shoreline.

1.3 Assumptions and limitations

A lack of available field data regarding the volumes, rates, origin and the manner of transport of coarse material in the vicinity of the fixed sand-bypassing system poses a major limitation to this study. Assumptions were made in order to obtain a volume and rate for the coarse material, but these values are not an exact representation of the actual conditions.

1.4 Study structure

The formatting of the study aims to guide the reader through the various investigations until a final conclusion is established. The structure used is as follows:

<u>Chapter 1: Introduction</u>

Introduces the topic of the study together with the objectives, limitations and overview.

<u>Chapter 2: Port of Ngqura</u>

Introduces and describes the Port of Ngqura, the location of the Port, as well as the relevant history of Algoa Bay.

Chapter 3: Sand-bypassing systems

Provides a detailed description of the sand-bypassing system at the Port of Ngqura with its current challenges, which will serve as the basis for this study. Other successful fixed sand-bypassing systems are also discussed.

Chapter 4: Sources of coarse material

Discusses the different sources of coarse material for the sandtrap.

Chapter 5: Principles of sediment transport and particle motion

Includes an overview of the foundation of sediment transport, different modes of transport, and the dynamics of sediment transport in the coastal region together with the depth of closure.

Chapter 6: Site description

Identifies important areas relating to the topic with a focus on describing the study site and sediment dynamics within Algoa Bay.

Chapter 7: Conceptual solutions

Creates conceptual solutions based on the knowledge obtained in the previous sections.

Chapter 8: Revetment and sandtrap modification.

Describes the required modifications for the revetment and sandtrap that were coupled with the conceptual solutions.

Chapter 9: Conclusions and recommendations

Discusses results from the preceding chapters in accordance with the study objectives. Recommendations are also incorporated.

Appendix A:

Explains the assumptions that were made to determine the depth of closure, submerged groyne and rock revetment.

Chapter 2: Port of Ngqura

2.1 Introduction

The Port of Ngqura is the newest deepwater port in South Africa and is also one of the largest projects of its kind in Africa. Strategically positioned in the global east-west trading route, the port offers world-class port facilities and global container shipping. It forms part of the Coega Industrial Development Zone, but falls under the jurisdiction of Transnet National Ports Authority (TNPA).

The physical layout of the Port was significantly influenced by the paleo-channel at the mouth of the Coega river (du Plessis, 2010). Paleo-channels are riverbeds created when sea levels were lower and that now lie buried beneath the seafloor, filled with aggregate (Sea Surveyor Inc., 2013)). The Coega paleo-channel enabled a deepwater port without the necessity to dredge large quantities of consolidated material, resulting in lower capital dredging costs.

The Port of Ngqura is sheltered by two breakwaters. On the eastern side, the main breakwater has a total length of 2610 m, making it the longest breakwater in South Africa. It is designed to withstand wave heights of up to 9 m (Transnet, 2010). The secondary breakwater, with a length of 1080 m, is located on the western side (as illustrated in Figure 1 below).

The Port of Ngqura is a ground-breaking project for ports in South Africa, and remains unique for two reasons: it is currently the only port in South Africa with a fixed sand-bypass system and environmental authorisation for its construction and operation (Transnet, 2010).

Figure 1: Breakwater layout of the Port of Ngqura (Google Earth, 2015).

2.2 Location

The Port of Ngqura is located in Algoa Bay on the Eastern Cape coast of South Africa, as depicted in Figure 2 below. The south coast of South Africa consists of a number of crenulated, half-heart or log-spiral bays, with Algoa Bay known as the eastern most and largest of them all (Goschen & Schumann, 2011). These bays serve as protection against the south-westerly winds as well as ocean waves from the Southern Ocean.

Figure 2: Algoa Bay location (Google Maps, 2015).

Algoa Bay is bound by two headlands known as Cape Recife (on the western boundary) and Cape Padrone (on the eastern boundary). The enclosed coastline extends for 90 km and includes 80 km of surf-swept beaches. These beaches and their associated surfzones vary widely in physical form owing to the combined effects of variations in coastal orientation relative to prevailing winds, deep-water swell and sheltering headlands (Woolridge *et al*., 1997).

The two major rivers located in Algoa Bay are the Swartkops and Sundays rivers, with the Coega river located between them. The Swartkops river is located on the western section of Algoa Bay, whereas the Sundays river is located to the northern side. The detail of Algoa Bay's layout can be seen in Figure 3 below.

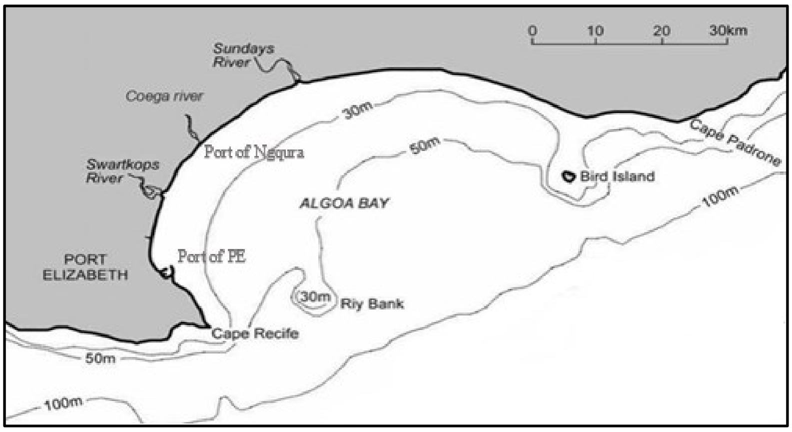


Figure 3: Layout of Algoa Bay (Schumann et al.*, 2005).*

Two ports are located in Algoa Bay. The first, the Port of Port Elizabeth (PE), was established in 1820 and is located on the western section of Algoa Bay. The second, the Port of Ngqura, is located in the mouth of the Coega river and situated a mere 20 km from the Port of PE (see Figure 3 above).

On the eastern section of Algoa Bay, the unique Alexandria Dunefield stretches for a distance of 50 km from the mouth of the Sundays river to Bushmans river, and is one of the largest existing active coastal dunefields in the world (Goschen & Schumann, 2011). The islands of St Croix, Brenton and Jahleel can be found a few kilometres offshore and are located between the Swartkops and Sundays river systems. Jahleel is located opposite the Port of Ngqura, and all of these islands occur inside the 30 m depth contour. The above mentioned rivers and dunefield play a vital role in the sediment dynamics of Algoa Bay, whereas some of the islands form part of animal conservation in Algoa Bay.

2.3 History of the Algoa Bay shoreline

To understand the present dynamics and conditions of the Algoa Bay shoreline, it is necessary to understand the changes that Algoa Bay has undergone in the past. As one might expect, the most drastic changes were made by man, in particular the construction of the Port of PE, and more recently the Port of Ngqura (Goschen & Schumann, 2011).

The (formal) coastal developments in PE started with the construction of a small breakwater in the early 1800s, which was shortly replaced by different types of jetties throughout the years. These structures never became permanent solutions because of the limited protection they were able to proffer incoming vessels (Goschen & Schumann, 2011).

Eventually, tougher protection measures were resorted to for Algoa Bay, with the first block of a breakwater laid in 1922 (Goschen & Schumann, 2011). It did not take long to realise the environmental consequences of this structure, however. With the accumulation of sand at the south-eastern side of the harbour forming a new beach (King’s Beach), the beach at the northern side, North End Beach, eroded rapidly. King’s Beach continued to expand over the years (see Figure 4 below) and eventually sand rounded the end of the breakwater and entered the harbour mouth. This caused the need for dredging in the harbour mouth channel at regular intervals.

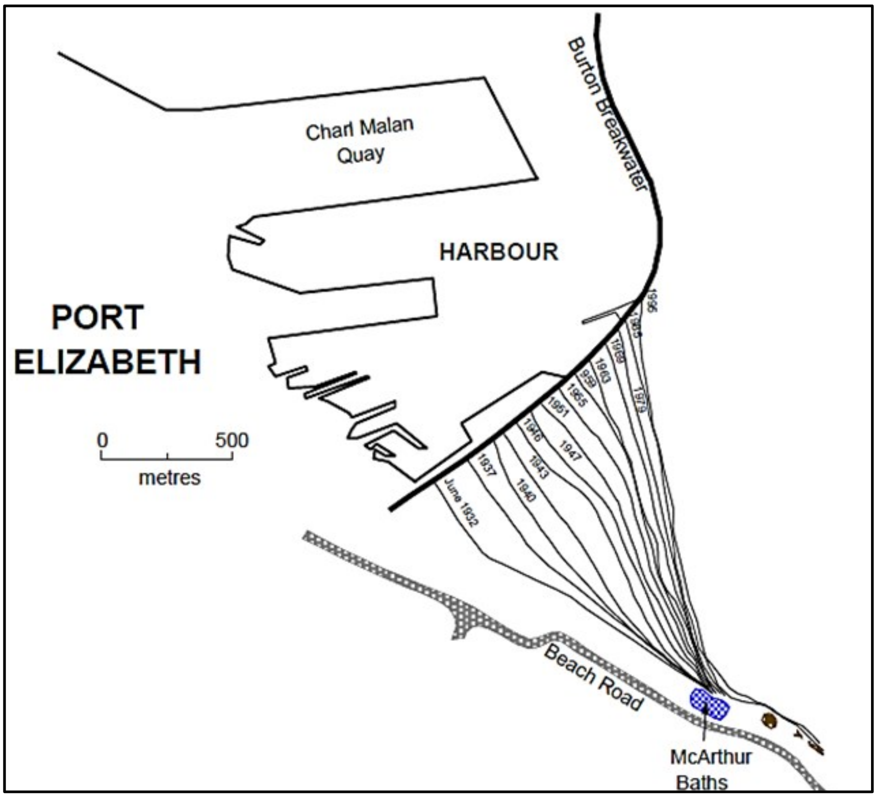


Figure 4: Accretion of King's beach 1932-1996 (Goschen & Schumann, 2011).

These developments were followed by the stabilisation of the dunes because of the dominant south-westerly winds blowing over Drift Sands; posing a major threat to the new harbour plans, covering tram tracks, and destroying rolling stock. The stabilisation was achieved through the planting of artificial vegetation, which prevented the movement of sand past this section (Goschen & Schumann, 2011).

The Council for Scientific and Industrial Research (CSIR) analysed changes to the Algoa Bay coastline from 1899 to 1969 and found that the coastline eroded inshore by more than 200 m in some places (Goschen & Schumann, 2011). A bypassing system was planned in order to restore some of the coastline prior to construction, but with major dredging constantly required to keep the harbour mouth open, it never became a priority.

Despite the environmental impact of these developments on the Algoa Bay coastline only increasing, the Port of Ngqura was completed a mere 20 km from the Port of PE; resulting in more environmental challenges. To help address the problem where the Port of Ngqura is concerned, a sand-bypassing system was designed to replicate the natural movement of sand past the harbour.

Until now, however, there have been numerous problems for the Port of Ngqura, causing the sand-bypassing system to only deliver a fraction of the required amount of sand (Rutherford, 2015). If these challenges are not addressed in the near future, Algoa Bay and its surrounding areas may face more environmental problems.

Chapter 3: Sand-bypassing systems

3.1 Introduction

The sand-bypassing system of a port is considered a key element in most modern portal projects, with the primary objective of such systems being to limit undesired environmental consequences.

One of the major environmental consequences of a port is the interruption of the longshore transport of sediment along the coastline. This interruption is caused by coastal structures extending seawards; obstructing the natural path of the sediment and leading to substantial accretion on the up-drift side and severe erosion on the down-drift side. If left unattended, bypassing may happen naturally but this usually forces sediment to travel into the entrance channel of a port; causing major navigation challenges (Gold Coast Waterways Authority, 2015).

The erosion on the down-drift side is caused by a lack of sand supply, which will have a major impact on the shape of the down-drift coastline. This process can be addressed by making use of a sand-bypassing system that transports sediment from the up-drift side to the down-drift side; allowing the sediment to continue on its natural course.

In this section, thorough research is conducted on the fixed sand-bypassing system installed at the Port of Ngqura in order to obtain a complete understanding of how the system works and where the problems are potentially to be found. Two other examples of such systems with similar environmental conditions are also investigated so that comparisons can potentially be made and lessons may be learned for the Port of Ngqura.

3.2 Port of Ngqura sand-bypassing system

Several types of sand-bypassing schemes were considered for the Port of Ngqura, but a fixed sand-bypassing system was installed at the Port. The layout of the entire the system is illustrated in Figure 5 below. In the following section, each component of the sand-bypassing system at the Port of Ngqura is described in detail.

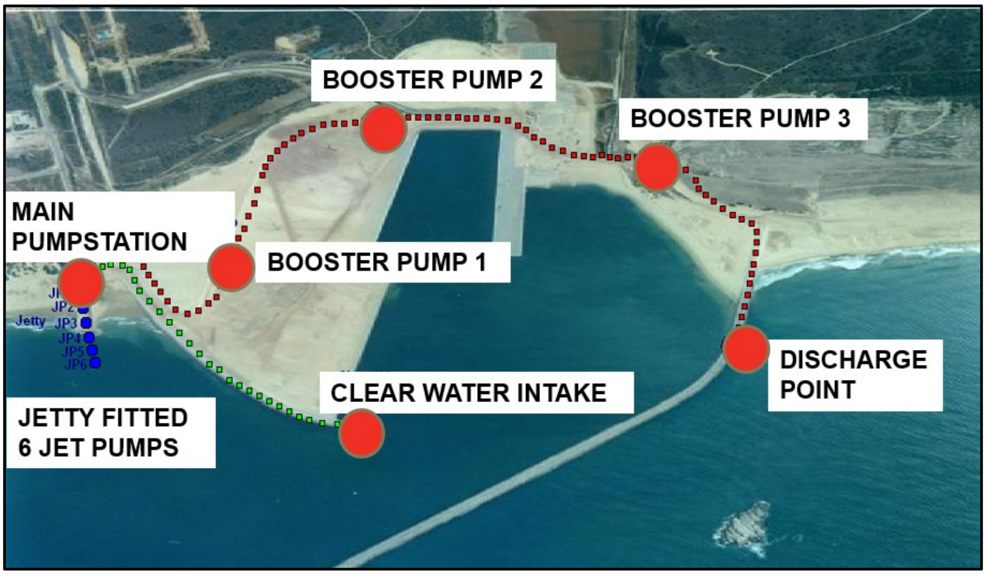


Figure 5: Layout of the sand-bypassing system components at the Port of Ngqura (Transnet, 2016).

3.2.1 Jetty and sandtrap

The sand-bypassing system at the Port of Ngqura consists of fixed jet pumps on a jetty in the surfzone. The 225 m-long jetty was constructed over an excavated sandtrap, which is 8m below chart datum (CD). The jetty is located 150 m from the western breakwater as shown in Figure 6 below.

Figure 6: Port of Ngqura sand-bypassing jetty (Transnet, 2016).

The purpose of the excavated sandtrap is not only to serve as a temporary storage area, but also to trap the longshore transport in a desired area where the jet pumps will be able to reach. The properties of the sandtrap are shown in Figure 7 below.

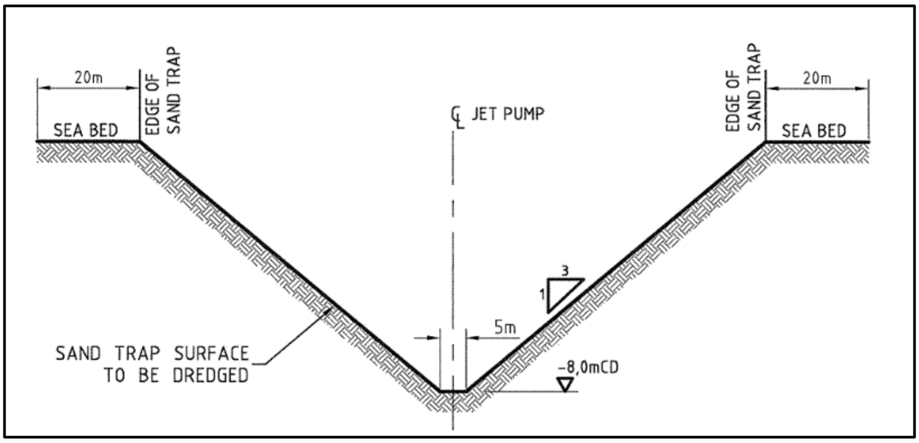


Figure 7: Sandtrap properties (Transnet, 2013).

3.2.2 Jet pumps

Six jet pumps are located on the jetty, positioned 27 to 36 m apart, with a design depth of -7.5 to CD. The jet pumps were designed to not only pump marine sand, but also cobbles of up to 150 mm. The sand slurry created by the jet pumps contains 12-15% sand. The layout of the jet pumps is illustrated in Figure 8 below.

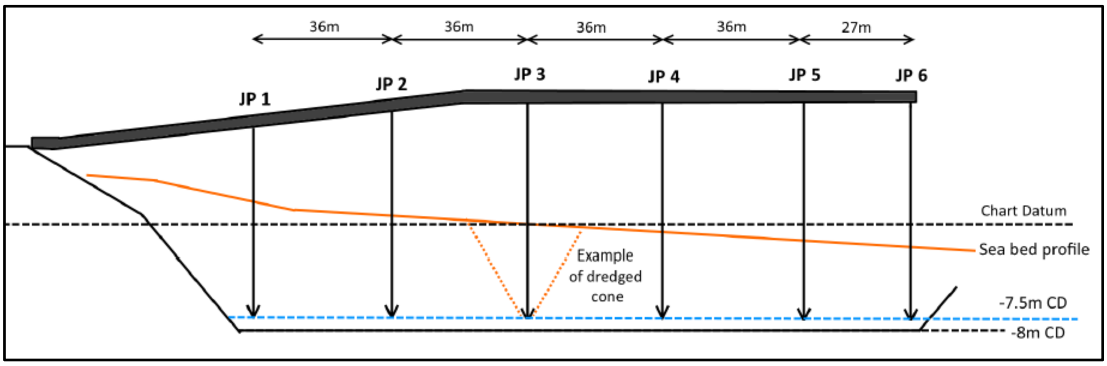


Figure 8: Layout of jet pumps at the Port of Ngqura (Rutherford, 2015).

3.2.3 Pump stations

The clear water intake is located at the end of the western breakwater, with main pump station at the landside of the bypassing jetty. There are also three booster pumps to help achieve the necessary pumping rate through a 4 km-long discharge pipe to the eastern breakwater.

3.2.4 Design capacities

The system was designed to bypass a target rate of 150 000 - 200 000 m^3 per year, with 150 000 m^3 being the lower limit, but it has sufficient capacity to bypass up to 320 000 m^3 per year (Transnet, 2016). During extreme events, such as a storm, the system was also designed to be able to bypass a capacity of 16 000 m^3 in five days. The pumping rate ranges between 110 and 250 t/hr of in-situ material, and depends on the grading of material being pumped, the motive water pressure and the position of the jet pump being operated.

3.2.5 Bypassing Process

The sediment located in the sandtrap is transported from the intake jetty to the discharge location in the form of a sand/water slurry. To create the slurry mixture, seawater is pumped from the clean water intake positioned within the caisson of the secondary breakwater to the main pump station at the base of the jetty.

The clean water is then pumped to the jetty jet pumps using a high pressure motive water pump and a low pressure fluidising/flushing water pump situated in the main pump station. The low pressure flow passes through fluidising nozzles to create a sand/water slurry fluid around the jet pumps. The fluid is then entrained into the high pressure flow, which passes through a mixing chamber. Low pressure flushing water is pumped through the nozzles to prevent an ingress of sand when the system is in idle mode.

The sand/water slurry discharged by the operational jet pump is pumped to the booster slurry pump located in the main pump station. A series of three further booster pumps positioned in satellite booster pump stations located within the port area provide the power required to pump the sand/water slurry to the eastern breakwater discharge location (see Figure 5 above).

3.2.6 Performance

The performance of the sand-bypassing system is measured by the quantity of sand bypassed annually. Table 1 below displays the total tonnage bypassed and mechanically transported (Transnet, 2016).

Table 1: Sand-bypassing system total tonnage for the system and mechanical transport combined (Transnet, 2016).

Month	2007	2008	2009	2010	2011	2012	2013	2014	2015	2016
January	0	15 247	0	0	6 293	2 015	2 015	19 884	4 506	0
February	0	15 109	0	5 750	4 096	4 566	0	15 658	11 535	703
March	0	9 615	11 086	34 580	6 014	15 507	3 872	15 248	51 444	21 966
April	0	9 914	21 131	19 885	8 997	42 068	5 865	24 332	27 268	15 581
May	0	17 088	19 275	0	3 029	36 353	17 331	1 040	29 188	16 400
June	0	11 035	23 171	0	5 260	5 774	14 099	15 798	26 610	11 224
July	15 302	16 954	1 301	2 124	2 186	0	19 415	1 744	33 022	
August	2 942	17 589	0	3	22 397	772	22 957	29 600	23 005	
September	4 226	8 547	0	2 253	6 922	8 006	14 613	23 146	27 138	
October	148	0	1 690	10 726	0	16 372	19 500	23 445	17 362	
November	1 072	0	2 598	11 035	0	4 724	46 044	26 014	16 706	
December	6 206	0	419	7 475	0	1 293	40 150	21 766	20 193	
Total	29 896	123 106	80 671	93 831	65 194	137 540	205 951	227 039	287 977	65 894
Total bypassed	1 317 379									

Table 1 above indicates that there are several months with no rate of delivery, indicating that operation was halted during that period. Rutherford (2015) estimated that the relationship between dry sand in tonnes with respect to in-situ material in volume is equal 1 to 1.6 for the Port of Ngqura. The values in Table 1 are converted to the annual volume per year as shown in Table 2 below.

Table 2: Annual volume bypassed.

Year	2007	2008	2009	2010	2011	2012	2013	2014	2015	2016	Total
Volume (m^3)	18 803	77 425	50 736	59 013	41 003	86 503	129 529	142 792	181 118	41 443	828 364

The annual volumes from Table 2 were used to produce Figure 9 below, which illustrates the annual volume bypassed from 2007 to 2016. The minimum target volume of the system is also indicated in Figure 9.

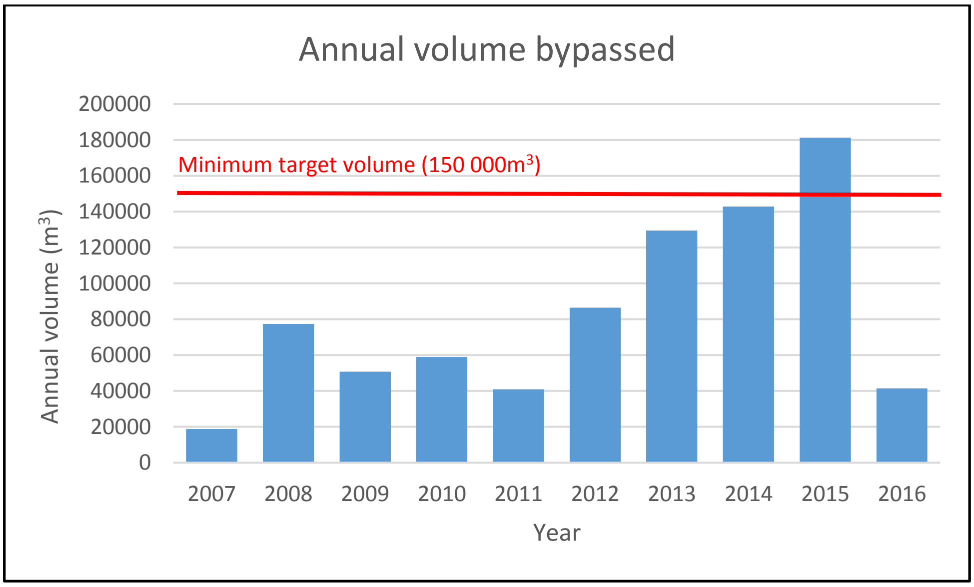


Figure 9: Annual volume bypassed.

Figure 9 shows that over the past nine years the minimum target volume was only achieved in 2015 and that the total quantity bypassed is only 61 % of the minimum required target volume. Although the sand-bypassing system shows vast improvement in the past three years, several additional actions were taken to ensure that the bypassing system's performance improves. These actions are only temporary solutions and will be discussed in Section 3.2.9. The reasons why the required rates were not achieved were described by Transnet (2016) as the following:

- lack of replenishment of the sand in the sandtrap;
- the sandtrap is filled with coarse material;
- only three of the six jet pumps are operational;
- all functional jet pumps were elevated from -7.5 m to CD to -4.1 m to CD; and
- the system is often not operating due to breakdown.

3.2.7 Current challenges

Several factors are influencing the work rate of the system, with the primary one being the migration of coarse material into the sandtrap. Over time, the sides of the crater at the jet pumps become armoured with coarse material, reducing the flow of sand towards the jet pumps and ultimately preventing the fluidisation of sand and pumping thereof. Without the fluidisation of sand, limited sand can be bypassed and the design rate cannot be achieved.

The production of two of the six jet pumps has been restricted due to the sandtrap being filled with coarse material, which obstructs the intake of the jet pump as shown in Figure 10 below. The previous jet pump maintenance resulted in the jet pumps being dredged out from a depth of -7.5 m to CD for inspection and service purposes (Jansen, 2015). After the process was completed, the jet pumps were unable to be dredged back to the design level, which resulted in four pumps operating at -4.1 m to CD (Jansen, 2015).

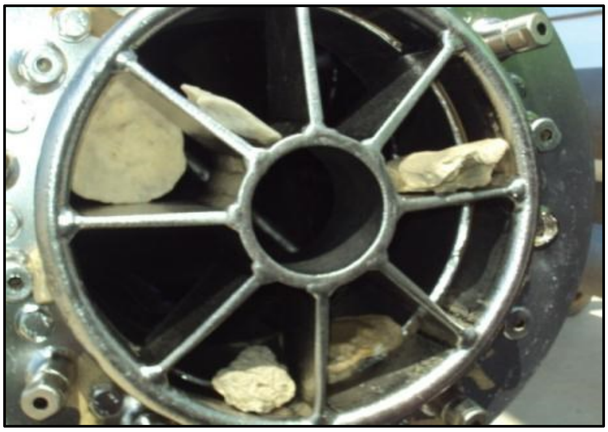


Figure 10: Obstructed jet pump intake (Transnet, 2016).

Largely as a result of this process, the system can only operate four of the six jet pumps. Jet Pump 5 is trapped by stone, and cannot be lifted out for cleaning, while Jet Pump 6 cannot be lowered to its full depth due to the accumulation of coarse material in the sandtrap (Jansen, 2015).

3.2.8 Maintenance

The Port of Ngqura follows a maintenance schedule lasting one week and performed every three months, which includes:

- the removal and installation of pumps with a crane for inspection and removal of marine growth;
- high pressure cleaning of pipe lines;
- divers removing intake screens and marine growth in the caisson, and re-installing the intake screen; and
- daily removal of coarse material from the beach (sizes exceeding jet pump intake size).

This entire process takes approximately one week to complete, which means the bypassing process is halted for this period.

3.2.9 Current solutions

In 2008, 44.5 m^3 of debris was removed along the full length of jetty. The material consisted of 57% naturally-occurring debris (beach rock and pebbles, etc.) and 43% temporary works remnants (quarried rock, blue bags bentonite, etc.). A total of 69% percent of debris was located between Jet Pump 1 and Jet Pump 3.

In 2015, another dredging process commenced at Jet Pump 1 and 2. The smaller rocks were removed by a dredge pump with a maximum size of up to 100 mm as shown in Figure 11. The larger rocks were airlifted from the sandtrap, reaching a size of up to 300 mm as shown in Figure 12. The rocks that were physically removed varied from 50 mm to 450 mm.

Figure 11: Dredge pump used during dredging works (Transnet, 2016).

Figure 12: Rocks airlifted from sandtrap (Transnet, 2016).

A process to elevate the jet pumps above the accumulated debris at the base of the cone was applied to increase the production. The jet pump bypasses the sand in the vicinity of the intake; as the sand is removed, the sand located above the removed sand fills volume. This means that by elevating the jet pumps above the accumulated debris, the volume of sand that is available to bypass is directly reduced.

This method functioned up to a certain point until all of the sediment in the vicinity of the jet pumps were bypassed. To resolve the issue, the Port used bulldozers to manually feed the cones. While effective, this is a costly procedure that cannot be regarded as a permanent solution.

In order to minimise the risk of blockage at the intake, a jet pump filter cage was designed, as shown in Figure 13 below. This method solved the obstruction problems at the intake, but resulted in a reduction of intake size, which meant that the intake size of the entire system decreased. This resulted in an even lower available volume that can be bypassed and just moved the obstruction to another location.

Figure 13: Jet pump intake cages (Transnet, 2016).

3.3 Other fixed bypassing systems

While several other types of methods are currently in use for sand-bypassing around the world, the choice of a bypassing system depends on the unique characteristics of a project. Whether a fixed bypassing system or making use of dredgers are chosen, the main objective of a bypassing system is to match the longshore transport rate and to minimise the environmental effect of a coastal structure. By referring to two other examples of such systems with similar environmental conditions, lessons may potentially be learned for the Port of Ngqura system.

3.3.1 Nerang river sand-bypassing system

3.3.1.1 Introduction

The Nerang river entrance is located along the mid-east coast of Australia along the Gold Coast region of Queensland (see Figure 14 below for the location of the Nerang river entrance). One of the major challenges hindering safe passage into the river was the strong northerly sediment transport at the location. It was estimated that the unstructured Nerang river entrance migrated north at an average of 60 m per year, causing land erosion and changing sandbanks at the bar (Gold Coast Waterways Authority, 2015). The channel was stabilised to help prevent this process.

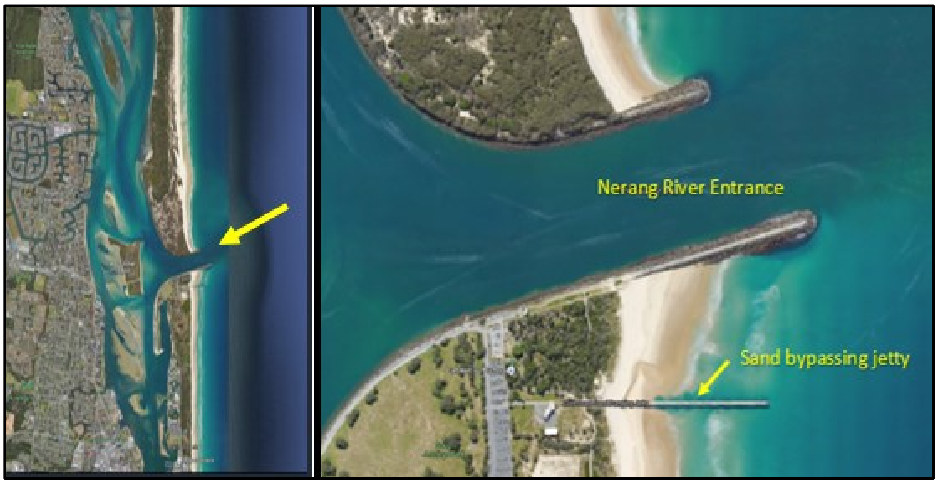


Figure 14: Nerang river entrance location (Google Earth, 2015).

The Nerang river project remains an exceptional case because it was the first project where the bypassing system was an integral part of the stabilisation project from the start. While several different bypassing schemes were originally considered, a jet pump system was ultimately selected.

3.3.1.2 System specifications

The system consists of a steel-framed jetty over a 270 m sandtrap with an overall length of 500 m extended seawards (see Figure 15 below). Ten submerged jet pumps are suspended from the jetty. Sand in the vicinity of the jet pumps is forced into the system in a slurry mixture and discharged onto the Southern Ocean beach on South Stradbroke Island. The system was designed to bypass 500 000 m^3 per year (Gold Coast Waterways Authority, 2015).

Figure 15: The Nerang river sand-bypassing system (Google Panoramic, 2015).

The Nerang river sand-bypassing system has been operating for 27 years and is still, as at 2014, considered a prime example of efficient sand-bypassing systems worldwide, with a total of 15 million m^3 of in-situ sand bypassed (Cowper & Thomas, 2014). Although this is more than the projected value of 500 000 m^3 per year, the system has not operated without problems.

3.3.1.3 System challenges

Over the years, the one major problem experienced by the Nerang river sand-bypassing system has been debris in the jet pump cones; interfering with the performance of the system. The debris, consisting of items such as rocks, bricks, wood and trash, find their way into the littoral system and end up at the bottom of the craters. Eventually these items restrict the flow of sand and the system performs at only 60% of the designed average rate (Clausner, 1989). This also increases the energy required for the system to operate.

While several approaches were tried to deal with the debris problem, the use of a clean-out jet pump was deemed the most successful. The jet pump was altered by enlarging the mixing chamber opening to 25 cm (as opposed to the 9 cm opening in a normal jet pump); enabling the pump to bypass larger debris. The other possible solution was the deployment of screens, which helped in the short term but still demanded frequent diver maintenance (Clausner, 1989).

The Nerang river bypassing system is still experiencing problems with the accumulation of larger material around the jet pumps. The current solution is to remove the jet pumps and to use a drag line with a clam shell bucket to remove the accumulated debris (Dean, 2002), which is a costly procedure.

3.3.2 Tweed sand-bypassing system

3.3.2.1 Introduction

The Tweed sand-bypassing system is located in the mouth of the Tweed river on the eastern coast of Australia, also known as the Gold Coast. The reason for installing a sand-bypassing system at this location was that safe passage into the Tweed river could not be guaranteed because of sand shoals that formed at the river entrance (Tweed Sand Bypass, 2016a). Although training walls were extended in order to improve navigation, a sand bar continued to form at the river entrance. As a result of the construction of training walls, accretion occurred at the up-drift side with a large sand build-up forming along the walls and severe erosion occurring along the southern Gold Coast beaches. Eventually, sand was building up to such an extent that it moved past the end of the breakwater and started to build up in the river entrance (Tweed Sand Bypass, 2016a).

3.3.2.2 System specifications

To solve the problem of sand build-up at the Tweed river entrance, a permanent sand-bypassing system with a target rate of 500 000 m^3 per year was constructed (Tweed Sand Bypass, 2016a).

The system comprises a sand-collection jetty with an overall length of 450 m, with a series of ten submerged jet pumps, constructed perpendicular to Letitia Spit beach some 250 m south of the southern breakwater (Tweed Sand Bypass, 2016a). The natural longshore transport fills the sandtrap on the up-drift side. The jet pumps produce a sand slurry from the entrapped sand and transport it to one of four outlets in the northern New South Wales and Southern Queensland beaches. This system decreases the amount of sand entering the river entrance and will reduce long-term erosion. A visual interpretation of the system can be seen in Figure 16 below.

Figure 16: Sand-bypassing system at the Tweed river (Tweed Sand Bypass, 2016a).

3.3.2.3 Performance

The Tweed sand-bypassing system has been running for more than 15 years with annual pumping quantities displayed in Figure 17 below.

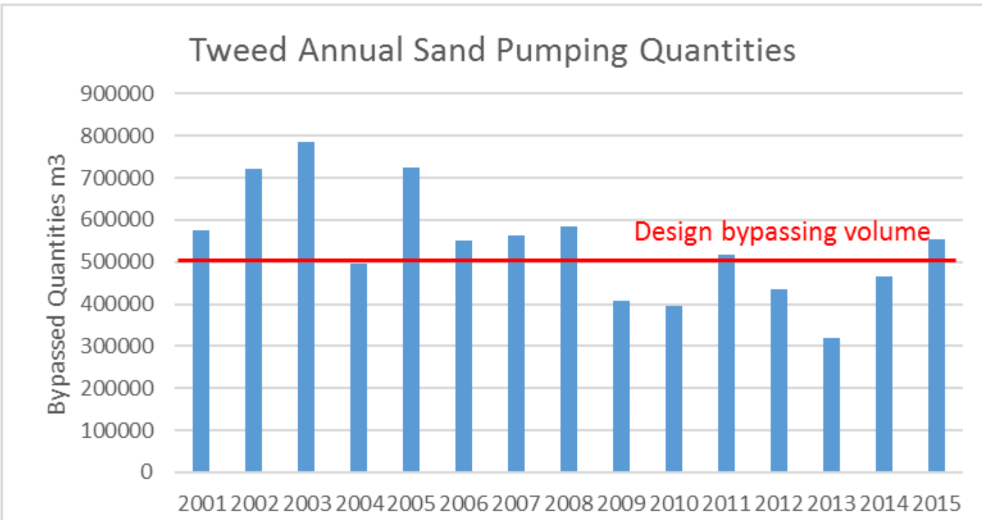


Figure 17: Tweed annual sand pumping quantities (Tweed Sand Bypass, 2016b).

At the end of 2015, a total quantity of 8 103 065 m^3 has been bypassed (Tweed Sand Bypass, 2016b), which is more than the expected 500 000 m^3 annually. Although the majority of the years exceeded the annual target rate, after 2008 the quantities decreased dramatically. While the system was designed for unattended operation, low operating costs and minimal maintenance requirements, an occasional mechanical breakdown and/or blockage still occur.

3.4 Conclusion

The sand-bypassing system at the Port of Ngqura has proven on several occasions that it can perform beyond its design capacity. Due to coarse material migrating into the sandtrap, however, the process is regularly forced to stop in order to do maintenance, which drastically reduces the production rate of the system.

The Port is making use of alternative methods in order to achieve the design rate of the sand-bypassing system, but these methods cannot be seen as permanent solutions. In order for the bypassing system to function according to the design specifications, the supply of coarse material to the sandtrap must be reduced.

The jet pumps of the sand-bypassing system can handle particles up to 150 mm, which means that all the coarse material with a diameter larger than 150 mm will cause an obstruction at the intakes. Therefore, it is necessary to determine the sources of these larger particles.

The two fixed sand-bypassing systems at the Tweed and Nerang river entrance are both located on the Gold Coast of Australia. Both these systems are designed to transport 500 000 m^3 per year and have thus far averaged above the target. However, the Tweed river bypass is exhibiting a downward trend in pumped volumes, but up until now there were no extreme measures taken to improve the performance. The Nerang river bypassing system is also experiencing similar problems with the accumulation of larger material around the jet pumps. Their current solution is to remove the jet pumps and to use a drag line with a clam shell bucket to remove the accumulated debris. However, this is a costly procedure which cannot be seen as a permanent solution for the Port of Ngqura.

Throughout this section it was shown that the accumulation of debris in the sandtrap seem to be a joint problem for all three these systems, which is why an alternative method must be found to address the problem at the Port of Ngqura.

Chapter 4: Sources of coarse material

4.1 Introduction

During a site visit to the Port of Ngqura in July 2016, it was noticed that the dredged material from the sandtrap appear to derive from multiple sources. Figure 18 below indicates the variety of coarse material removed from the sandtrap during previous dredging works. The properties of the coarse material gave an indication of the origin of the material. The origin of the majority of the sources are discussed in the following section.

Figure 18: Variety of the dredged material (Transnet, 2013).

4.2 Rock revetment

The main type of armour used in the breakwaters of the Port of Ngqura was concrete dolosse (a concrete block with a complex geometric shape used in great numbers to protect an area against the erosive force of ocean (Artefacts, 2014)). Towards the shore of the western breakwater, the armour layer was replaced by rock armour units because of the lower wave energy this section is required to withstand, which ultimately reduces the cost (Sorenson, 2013). The rock armour units were also used for the rock revetment located behind the jetty of the sand-bypassing system. Figure 19 below indicates the area where rock armour units were used at the Port of Ngqura.

Figure 19: Area of rock used for armour layer (Google Earth, 2016).

Both of these sections still experienced high wave action (during storm events), which caused some of the smaller units to loosen from their positions. Figure 20 below illustrates the movement of the armour units to a location beneath the jetty, which was not their original position.

Figure 20: Displacement of armour rock (Hack, 2015).

The wave actions caused the displacement of some of the armour units but several units also fractured because of the contact forces against one another. Figure 21 below shows the smaller fractions of the armour units, which have the same properties.

Due to the close proximity of the sand-bypassing system to the rock revetment, these smaller fractions tend to migrate into the sandtrap; producing an additional source of coarse material. The revetment was designed for armour units with granite properties. Therefore, it can be assumed that the coarse material with granite properties found in the sandtrap mainly originates from the rock revetment.

Figure 21: Armour rock breaking into smaller fractions (Hack, 2015).

The movement of the armour layer in certain sections of the revetment resulted in an exposed under layer. The under layer is the layer of units directly below the armour layer with the same properties but just smaller is size. This layer is not designed to withstand direct wave action, which is why they are smaller in weight. Through visual inspection, photos of the rock revetment and the opinion of the staff at the Port of Ngqura, it is assumed that these units migrate into the sandtrap.

Figure 22 below displays the state of the revetment behind the jetty. A distinction can be observed between the under layer (smaller units) and the armour layer (larger units) through the varying rock sizes at the bottom and top section. The smaller units from the under layer have a higher mobility, which increases the probability of these units ending up in the sandtrap.

Figure 22: Exposed under layer at rock revetment (Transnet, 2013).

4.3 Remnants of temporary construction works

The construction of the sand-bypassing jetty required significant temporary construction works to create a safe working platform, as shown in Figure 23 below. This was done by using various materials that included sand-filled geotextile bags and quarried rock. When the jetty was completed, the material was removed using mechanical grabbing and excavation. All of the material was not cleared during the operation, however, which resulted in significant quantities of temporary construction works remnants migrating into the sandtrap. It is evident that there is still a significant amount of temporary construction works remnants adjacent and in the sandtrap, specifically from jet pump 3 to 6 (Transnet, 2016).

Figure 23: Temporary works at the Port of Ngqura (Transnet, 2016).

4.4 Natural sources

During the site visit, the coarse material from the previous two sources was clearly identifiable because of the properties of the material. The majority of the remaining material was smaller in size and appeared to be more rounded in shape, as shown in Figure 18 above. The significance of this property will be explained in this section.

Beach pebbles are formed gradually over time as the ocean water washes over loose rock particles, which results in a smooth, rounded appearance (Geocaching, 2016). River pebbles are formed as flowing water washes over rock particles on the bottom and along the shores of the river, where they eventually end up in the coastal system (Geocaching, 2016). It is therefore assumed that these rounded units either experienced continuous wave action or originated from a river.

Through visual inspection, there also appeared to be some coarse material with coral origin that is assumed to be from a deteriorating reef within Algoa Bay. Although the majority of sediment in the longshore transport is sand, there is still a fraction of the longshore transport that consists of coarse material. Therefore, it can be concluded that the coarse material consists of river-derived material (river pebbles), material derived from local beach rock (beach pebbles), and from a coral origin.

These three sources end up in the longshore transport zone and gradually move along the coast until ending up in the sandtrap at the Port of Ngqura.

4.5 Conclusion

This section revealed that the coarse material located in the sandtrap at the Port of Ngqura originates from the rock revetment, remnants of the temporary works and natural sources.

The origin and properties of the coarse material supplied by the rock revetment and the temporary construction works is known, which means that enough information is available to create a solution for both these sources. The method to prevent the migration of coarse material from these two sources will be explained in Chapter 8.

The natural source of coarse material must, however, first be further investigated in order to obtain a better understanding how the entire system functions. Therefore, in Chapter 5 and 6 the focus shifts to the principles of sediment transport and particle motion as well as the origins, volumes and properties of this natural source.

Chapter 5: Principles of sediment transport and particle motion

5.1 Introduction

Although physical processes of sediment transport in tidal environments are extremely complicated and tend to be influenced by numerous factors, the basic principles of sediment transport in rivers and other environments may arguably offer valuable lessons for coastal environments. In this section, therefore, the general principles and conditions that cause sediment to move in a certain manner in a coastal environment are discussed.

5.2 Grain size

Since one of the most important parameters controlling the transport and deposition of sediments is grain size, it is convenient to classify sediment particles on this basis (The Open University, 1999). Sediments are classified into three main groups: mud, sand and gravel; each with a specific grain size limit. The most widely-used classification system for grain size is the Wentworth scale, as shown in Table 3 below.

Table 3: Classification of sediment particles (Wentworth, 1992).

Classification	**Particle diameter (mm)**	**Group**
Boulder	>256	*Gravel*
Cobble	64-256	
Pebble	4-64 m	
Gravel	2-4	Sand
Very coarse sand	1-2	
Coarse sand	0.5-1	
Medium sand	0.25-0.5	
Fine sand	0.125-0.25	
Very fine sand	0.062 0.125	
Silt	0.004-0.062	Mud
Clay	<0.004	

In Section 3.2.2 it was mentioned that the jet pumps are able to bypass particles with a diameter of up to 150 mm. Table 3 above indicates that the particles that will cause obstructions at the intakes, namely particles larger than 150 mm, are classified as cobbles or boulders. Throughout the study these two classes will be referred to as cobbles.

5.3 Modes of transport

In a coastal region, sediment on the surface will only move if the water flows fast enough over a particle to force it to move. Generally, not all sediment particles in a study area are of the same size, causing particles to move in a different manner.

5.3.1 Bedload

Bedload can be described as the part of the sediment transport where particles stay in regular contact with the seabed usually in the form of sliding, rolling or saltation. Sliding and rolling particles (see Figure 24 below) remain in continuous contact with the seabed, merely tilting as they move, whereas saltating (leaping) particles 'jump' along the seabed in a series of low trajectories (see Figure 24 below). These type of movements occur when the seabed's shear stress exceeds a critical value. Through visual observations, it has been suggested that a bedload particle will usually move within a region of less than 10 to 20 particle diameter height (Chanson, 2004). This method of transport is usually coupled with either low flows and/or large grains as shown in Figure 26 below.

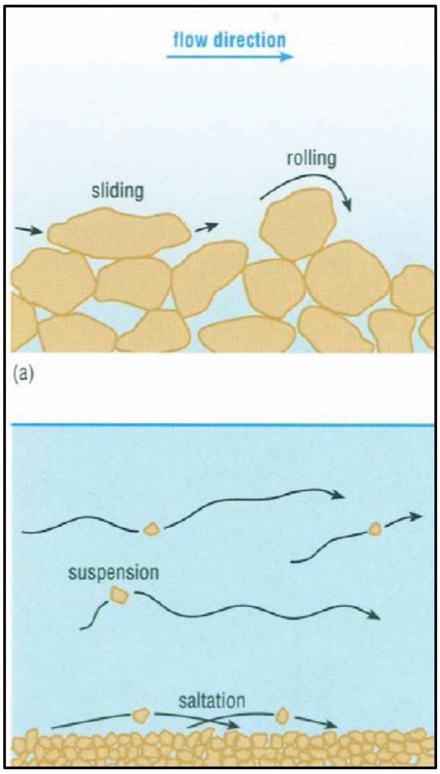


Figure 24: Sliding, rolling and saltation movements for bedload particle (The Open University, 1999).

5.3.2 Suspended load

The suspended load can be described as the part of the sediment transport where sediment particles are carried along within the water column, meaning that they follow a long and irregular path within the water and seldom come in contact with the seabed; until they are deposited when the flow diminishes (The Open University, 1999).

The sediment particles are sustained by turbulence and concentration profiles develop, while the balance between sediment settling and upward sediment diffusion from turbulence regulates the process. Figure 25 below illustrates the suspended concentration profile above the seabed.

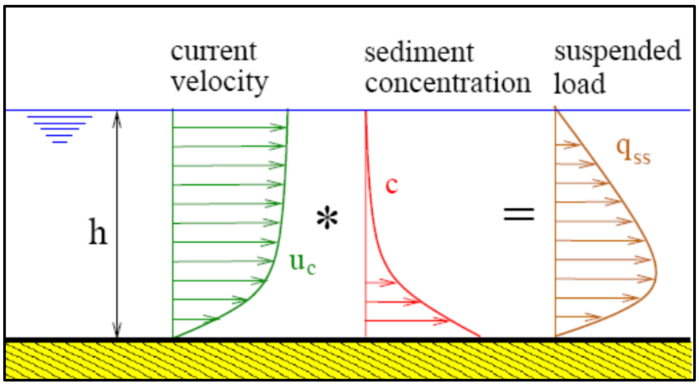


Figure 25: Current velocity, sediment concentration and suspended load profiles (Lund University, 2014).

As illustrated in Figure 25 above, the suspended concentration decreases with height above seabed; with the maximum concentration found just above the seabed. The suspend load maximum is located a distance above the seabed. This means that although sediment moves in suspension, the bedload and suspended particles move in close proximity from one another, resulting in the majority of the particles moving close to the seabed. Suspended load is usually coupled with either faster flows and/or small grains (The Open University, 1999), as illustrated in Figure 26 below.

5.4 Incipient motion

The initiation of motion, also known as incipient motion, is the initial movement of a particle that will occur when the instantaneous fluid force on a particle is just larger than the instantaneous resisting force related to the submerged particle weight and the friction coefficient (Nielsen, 1992). The position of the grain with respect to the surrounding grains (smaller particles hiding between larger particles) is also of significance because it will determine the degree of exposure of the grain, which is an important parameter determining the forces at the initiation of motion (Van Rijn, 2007).

The initiation of motion is a very important aspect when it comes to sediment transport as it will indicate the different velocities required to initiate movement for different size particles and whether or not a particle will start to move under certain conditions. In Figure 26 below, incipient motion for different particle sizes are indicated. It is clearly noticeable that particles with larger diameters require a larger velocity to initiate movement and vice versa. This implies that when a small velocity is applied to a study area, only the smaller particles will start moving, whereas a large velocity will move both large and small particles.

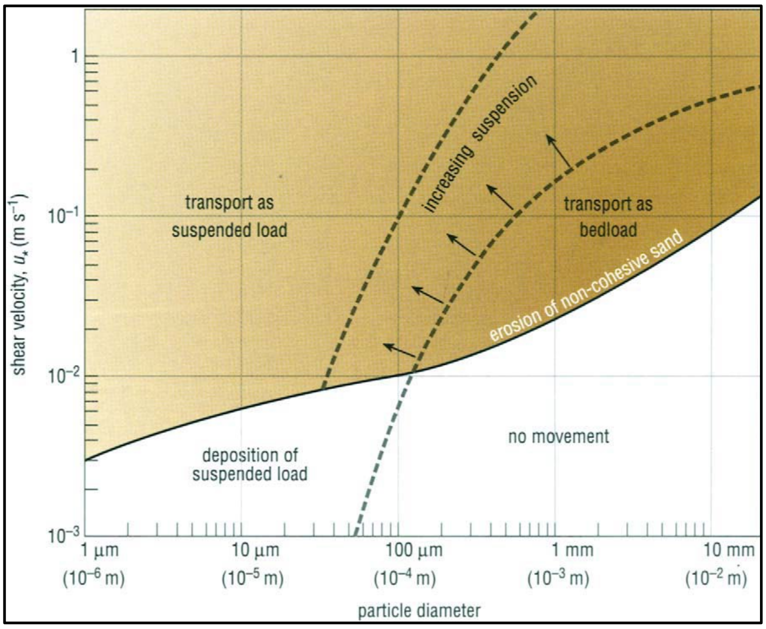


Figure 26: Incipient motion and methods of transport (The Open University, 1999).

5.5 Settling velocity

Sediment settling is also known as the fall velocity or terminal velocity, and reflects the moment when a particle falls through water and the acceleration is halted and continues to fall at the same velocity. This is caused by the drag force on the particles equalling the downward gravitational force. The falling velocity is represented in Equation 1 below which is based on Stokes law.

Equation 1: Falling velocity for sediment (Stokes, 1851)

$$v = \frac{1}{8} g \frac{(\rho_1 - \rho_2)}{\mu} d^2$$

- v = falling velocity (m/s)
- g = gravitational acceleration (m/s^2)
- ρ_1= density of particle (kg/m^3)
- ρ_2= density of fluid (kg/m^3)
- μ= fluid viscosity (m^2/s)
- d= particle diameter (m)

As indicated in Equation 1 above, the falling velocity is proportional to the diameter of the particle. A larger particle diameter will result in a larger settling velocity; which means that the particle will settle quicker. This implies that a small particle such as sand will spend a longer time in suspension compared to a larger particle such as a cobble.

5.6 Coastal sediment transport

In coastal environments, sediment transport is affected by waves, currents and undertow. The surfzone (where waves break) is the most active and unstable area of the coastal zone (PIANC, 2015). A mass of water moves over the sediment on the seabed, causing sediment to destabilise because of the strong shear that develops near the bottom.

Typically, littoral transport is assumed to act along two main directions, one parallel to the coast (long-shore) and one perpendicular to it (cross-shore), as illustrated in Figure 27 below. While these two sediment processes are both addressed separately in the next sections, it must be kept in mind that they function together. The cross-shore currents are mainly responsible for bringing the seabed grains to incipient motion, while the longshore currents are the main driver behind sediment transportation along the coast (PIANC, 2015).

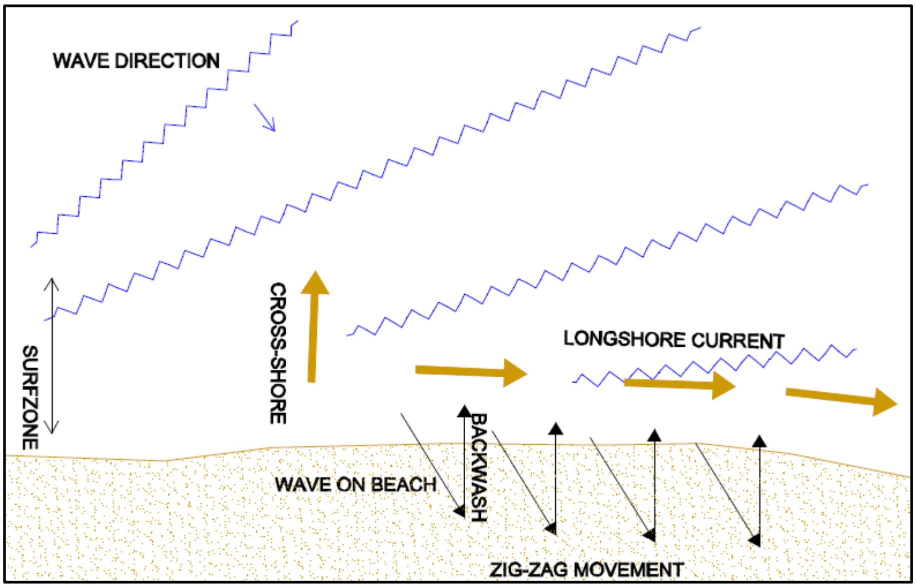


Figure 27: Cross-shore and longshore transport (PIANC, 2015).

5.6.1 Longshore sediment transport

Longshore transport is the total amount of material moved along the shoreline and can be related to the amount of energy/power available in the waves arriving at the shore (PIANC, 2015). These arriving waves induce longshore currents that are dependent on the wave heights and the incident wave angle (the angle at which the wave approaches the beach) relative to the bottom contours.

There are two different classifications for longshore transport, namely bulk and detailed transport. Bulk transport is a single value that represents the overall longshore transport across the cross-shore profile. The detailed transport, in turn, represents the different rates at diverse depths along the cross-shore profile, as illustrated in Figure 28 below (Schoonees & Theron, 2002).

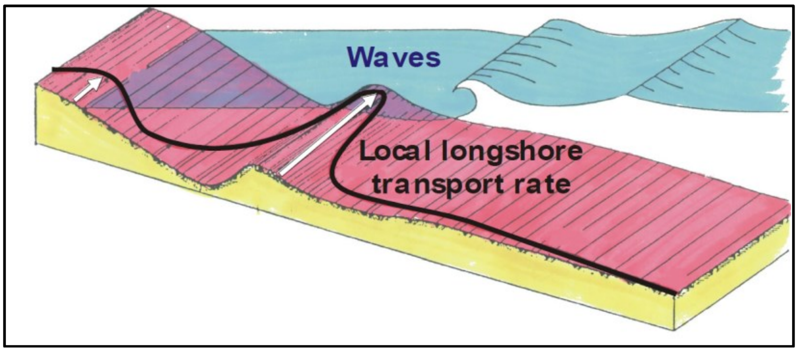


Figure 28: Detailed longshore transport profile (Schoonees & Theron, 2002).

Figure 28 above shows that longshore transport is dominated in the surfzone where wave-breaking and wave-induced currents prevail, whilst considerably less transport occurs seaward of the breaker point, up to the point where it stops. This point is known as the depth of closure, which will be discussed in Section 5.7.

The other important aspect to take note of is that there is a certain point in the cross-shore profile where the longshore transport is at a maximum. Dean & Dalrymple (2004) argue that this point is located in the breaker line and the midpoint of the surfzone. Kraus *et al.* (1982) suggest that this pronounced peak can be found in the swash zone.

5.6.2 Cross-shore sediment transport

PIANC (2015) argues that cross-shore transport is mostly caused by waves' mass transport and undertow. There are two type of cross-shore sediment transport, namely offshore transport, which occurs during energetic 'winter' periods, and onshore transport, which is more prevalent during mild wave activity. With these two types of transport dominating certain periods of the year, the cross-shore profile consequently fluctuates during the seasons of the year as depicted in Figure 29. The summer profile clearly shows that sands move towards the shore, whereas in winter the sand moves offshore.

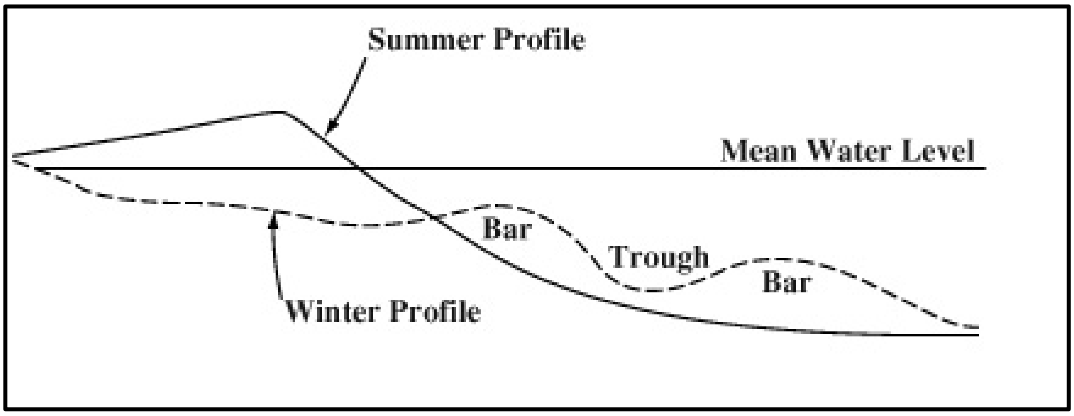


Figure 29: Cross-shore seasonal profile (Purkis & Klemas, 2011).

5.7 Depth of Closure

The depth of closure is a theoretical depth along a beach profile where sediment transport is very small or non-existent, dependent on wave height, period, and sediment grain size (U.S Army Corps of Engineers, 2012). Because the depth of closure is dependent on the sediment grain size, as mentioned above, the depth of closure will be discussed in two sections: the first will be the depth of closure for the longshore transport, and the second will be the depth of closure for cobbles larger than 150 mm.

5.7.1 Depth of closure for longshore transport

Rutherford (2015) recently did a study about the longshore transport at the Port of Ngqura. One of important aspects affecting the results of his model was the depth of closure, which was used to find the limit up to which sediment transport would occur. Dean & Dalrymple (2004) modified the original Hallermeier (1981) and Birkemeier (1985) equations in order to take a time frame other than one year into consideration. The modified version used to model the specific section is shown in Equation 2 and 3:

Equation 2: Depth of closure (Hallermeier,1981).

$$d_t = 2.28H_{e,t} - 68.5\left(\frac{H_{e,t}^2}{gT_{e,t}^2}\right)$$

Equation 3: Depth of closure (Birkemeier, 1985).

$$d_t = 1.75H_{e,t} - 57.9\left(\frac{H_{e,t}^2}{gT_{e,t}^2}\right)$$

- d_t = predicted depth of closure over time t (m)
- $H_{e,t}$ = non-breaking significant wave height that exceeds 12 hours per time t (m)
- g = gravitational acceleration (m/s^2)
- $T_{e,t}$ = wave period (s)

The average of both equations were used by Rutherford (2015) to determine the depth of closure, which was further calibrated to produce Figure 30 below. As illustrated in the Figure 30, the final result for the depth of closure was at 14 mean sea level (MSL), which is located 500 m seawards (Rutherford, 2015). This means that sand will move up to 500 m offshore.

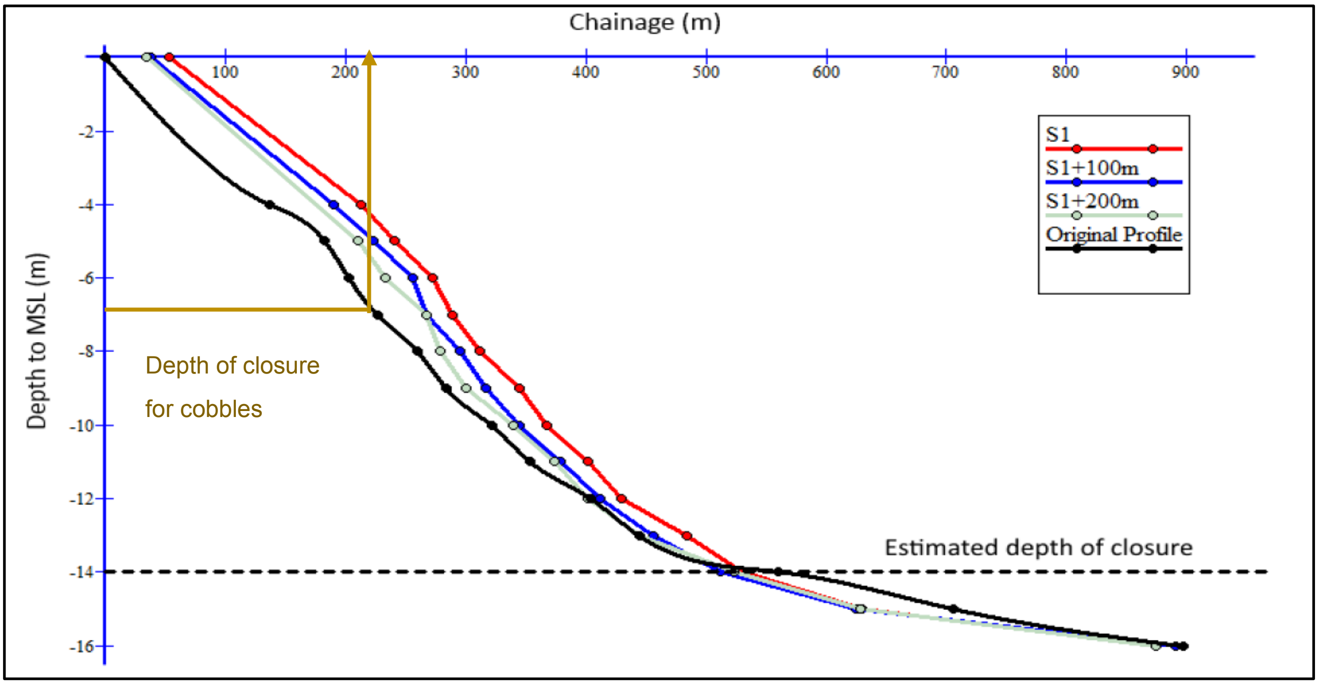


Figure 30: Depth of closure for sand (Rutherford, 2015).

5.7.2 Depth of closure for cobbles

In the previous section, the equations that were used for the depth of closure solely relate to the wave parameters, irrespective of sediment diameters. Therefore, a new depth of closure must be determined in order to know up to what depth cobbles will move.

While several methods are available to determine the depth of closure (with the sediment size as a parameter), two of the most frequently used theories include incipient motion and the beginning of longshore transport. Incipient motion focuses on movement in general, which can be in any direction.

The beginning of longshore transport indicates the position/depth at which longshore transport starts in the cross-shore profile, whereas the depth of closure indicates the depth at which sediment stops moving. Both these concepts indicate a depth where no further sediment transport will occur. Therefore, the beginning of longshore transport can be taken as the depth of closure.

Schoonees & Theron (1993) conducted a study on existing methods to determine the beginning of longshore transport. These methods were adapted in order to have the significant wave height and depth as parameters as shown in Equation 4 below. The relationship between the wave height and depth is determined by assuming a depth-limited state which will be used in Equation 5-10 below.

Equation 4: Significant wave height with respect to depth (Riedel et al, 1986).

$$H_{bs} = \frac{{H_s}^2}{\gamma d}$$

The seven methods that were used to determine the depth of closure is shown in Equation 5 to Equation 10. Every method has a specific critical value (f_{cr}). If the results from an equation is larger than the specific critical value, it implies that longshore transport is occurring at that specific depth.

The depth at which Equation 5-10 is equal to the critical value of that equation is taken as the beginning of longshore transport, which, as stated previously, is depth of closure. The assumptions that were made for these equations are shown in Appendix A.

Equation 5: Van Hijum-Pilarczyk-Chadwick (1977, 1982, 1989).

$$f_{vhcp} = \frac{H_{bs} cos\theta_b{}^{0.5}}{D_{90}}$$

- $f_{cr,vhcp}$ = 8.3
- H_{bs} = significant wave at breaking point (m)
- θ_b= angle of incidence (°)
- D_{90} = grain size with 90% of the grains sizes are smaller (m)
- Subscript vhcp- refers to Van Hijum-Pilarczyk-Chadwick

Equation 6: Brampton-Motyka-Chadwick (1984, 1989).

$$f_{cr,bmc} = \frac{8.1 D_{90}}{H_{bs}}$$

- f_{bmc}= 1.0
- Subscript bmc- refers to Brampton-Motyka-Chadwick

Equation 7: Chadwick (1989)

$$f_c = P_{lrms} = 0.5 P_{ls} = 0.5\left(0.125 \rho g H_{bs}{}^2\right) n C_b sin\theta_b cos\theta_b$$

- $f_{cr,c}$= 12.2
- ρ = seawater density (kg/m^3)
- g = gravitational acceleration (m/s^2)
- nC = group velocity (m/s)
- Subscript c - refers to Chadwick

Equation 8: Modified Morfett (1990).

$$\frac{d}{H_s} = \frac{1}{0.24 log_{10}(1000 D_{50})}$$

- d = waterdepth (m)
- D_{50} = median grain size (m)

Equation 9: Van der Meer-Veldman (1990, 1992).

$$f_{vdm} = \frac{H_{bs} cos\theta_b{}^{0.5}}{D_{n50}}$$

- $f_{cr,vdm}$= 11
- D_{n50} = nominal grain size diameter (m)
- Subscript vdm- refers to van der Meer

Equation 10: Burcharth-Frigaard (1987, 1988).

$$\frac{H_s}{\Delta D_{50}} = 3.0$$

- $\Delta = \frac{\rho_s}{\rho_w} - 1$ (CEM, 2006)

The graph that was used for the Hydraulics Research method (n.d) is also shown in Appendix A. The input variables for these seven methods are H_s, T_p, D_{50} and ϴ, where H_s is the wave for the extreme events, T_p is peak wave period, which is usually associated with H_s, D_{50} is the median grain size, and ϴ is the angle of incidence.

The site conditions at the Port of Ngqura will be thoroughly discussed in Chapter 6, but the wave conditions are required for the equations above. Based on a study done by PRDW (2001), it was established that the wave conditions at the breaker zone in the region of the sand-bypassing system is represented by the wave rose depicted in Figure 31 below.

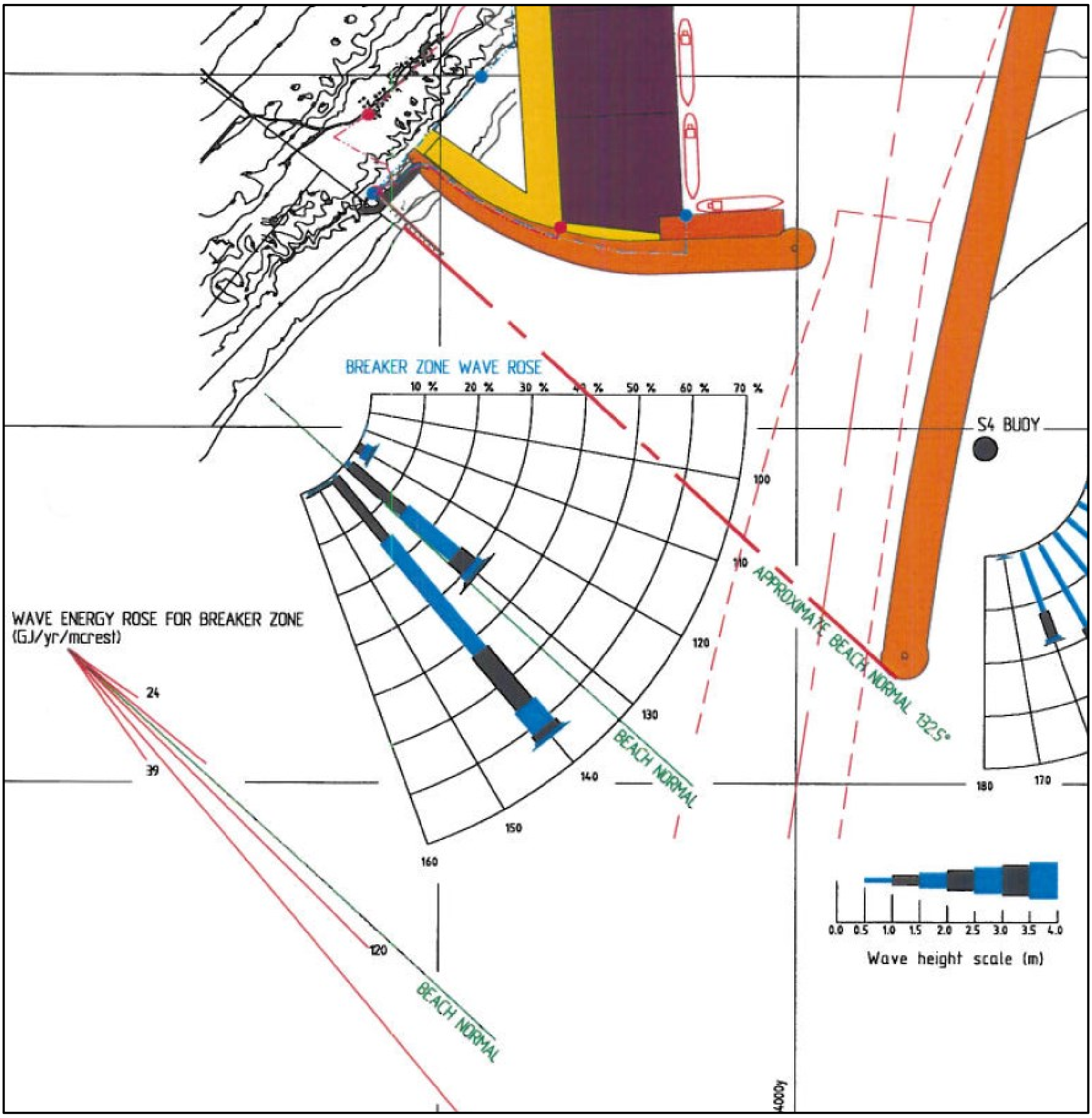


Figure 31: Wave conditions at the sand-bypassing system (PRDW, 2001).

The reason why the wave conditions in Figure 31 can be used rather than a wave conditions with a certain return period, is because this is not for the design of coastal structure, which means failure cannot occur. This means that the depth of closure value will have no direct threat to human safety or economic factors and the only aspect it will affect is the efficiency of the conceptual solutions, which will be discussed in Chapter 7. Therefore, the wave height is taken as the average of the highest wave heights in each direction of the wave rose.

The wave heights from the wave rose were used to find the corresponding wave period from a dataset comprising of three-hourly directional wave measurements in Algoa Bay deployed at a position approximately 100 m from the head of the breakwater of Port of Ngqura. The median diameter was taken as the 150 mm, which was the maximum size that the jet pumps is able to bypass, as explained in Section 3.2.2. The results obtained with these seven methods are indicated in Figure 32 below.

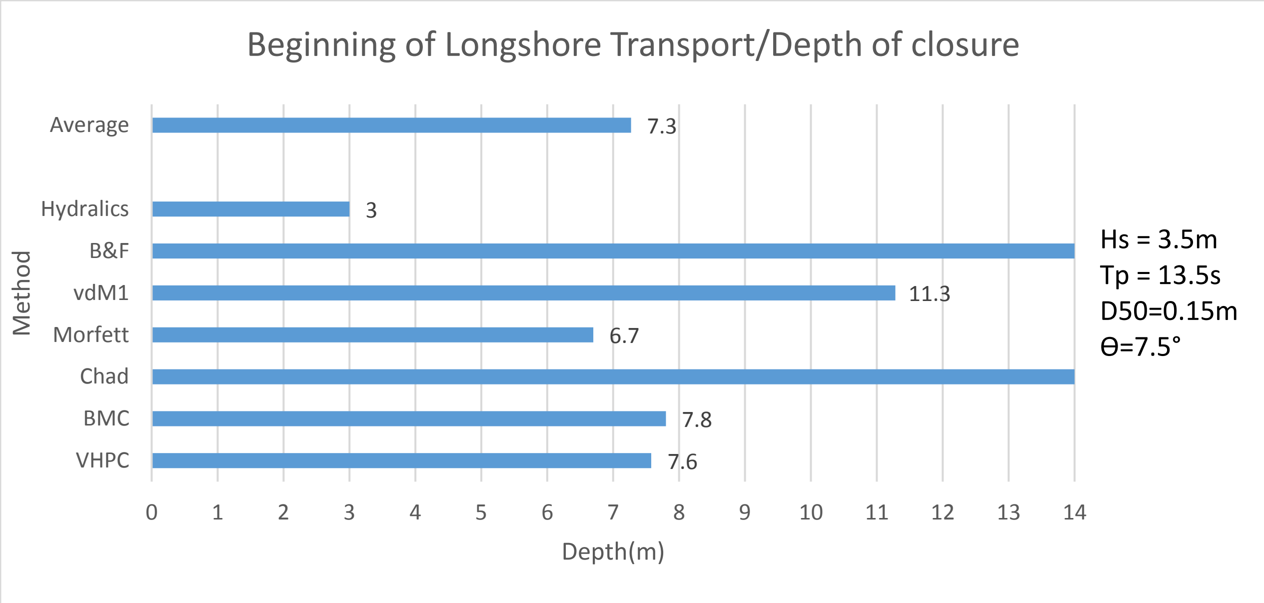


Figure 32: Depth of closure for cobbles.

The average of the results above will be used because the seven methods yielded very different results. Therefore, it is not known which method will serve as the best approach. The results obtained with the methods of Burcharth-Frigaard and Chadwick delivered results which were implausible, so these results were omitted when the average was determined. Schoonees & Theron (1993) experienced the same results with these two methods. The final results delivered a depth of closure of 7 m during storm wave conditions, at a location about 230 m seawards from the MSL datum (see Figure 27 above).

5.8 Conclusion

The most important concept derived from this background section is that different particle sizes have different characteristics when it comes to sediment transport in coastal regions. It has been shown that cobbles are more likely to move as bedload transport, whereas smaller particles (for example sand) will more likely move in suspension. Although these two methods of transport differ greatly from one another in concept, theoretically, the majority of sediment in motion will move near the seabed.

This aspect will be of great importance for the conceptual solutions in Chapter 7, because this means that if a structure is designed to obstruct cobbles, it will only be required to extend along the seabed to obstruct the bedload particles. However, theoretically, this structure will also obstruct the majority of the sediment in the longshore transport, which will be a critical aspect in the design of how to reduce this aspect.

The other two factors from this section that are of interest to the current study include, first, the longshore transport rate fluctuate at different depths in the cross-shore profile and at a specific position of the cross-shore profile there will be a peak. This aspect will be discussed in Chapter 6. Second, the incipient motion of a smaller particle (like sand) is lower compared to a larger particle (like cobbles); which means that cobbles require higher velocities to set and keep them in motion. It is well known that cobbles of this size will most likely move during storm events when the wave action is higher, resulting in higher velocities (Chadwick *et al.,* 2004)

The depth of closure for sand was determined by Rutherford (2015) as 14 m. The average depth of closure from the seven methods for cobbles with a diameter of 150 mm, delivered a depth of 7 m during storm wave conditions. These results showed that if a protection measure is used in order to obstruct the cobble movement in the surfzone, the structure will only need to extend up to a depth of 7 m instead of 14 m. This 7 m difference will not only mean a significant reduction of the costs involved to construct the structure, but will also reduce the impact of the structure on the shoreline. This is because a shorter length will obstruct a smaller volume of the longshore transport.

Chapter 6: Site description

This section contains a detailed description of the shoreline characteristics, sediment dynamics and site conditions within Algoa Bay. Each section will provide relevant information that will be used for the conceptual designs in Chapter 7 and the revetment modifications in Chapter 8.

6.1 Physical shoreline characteristics

The shoreline of Algoa Bay differs so substantially from one side to the other that the characteristics of the shoreline can only be described in sections and not as a whole. The information in this section was primarily derived from Theron (2014). Figure 33 below illustrates the different sections of the Bay.

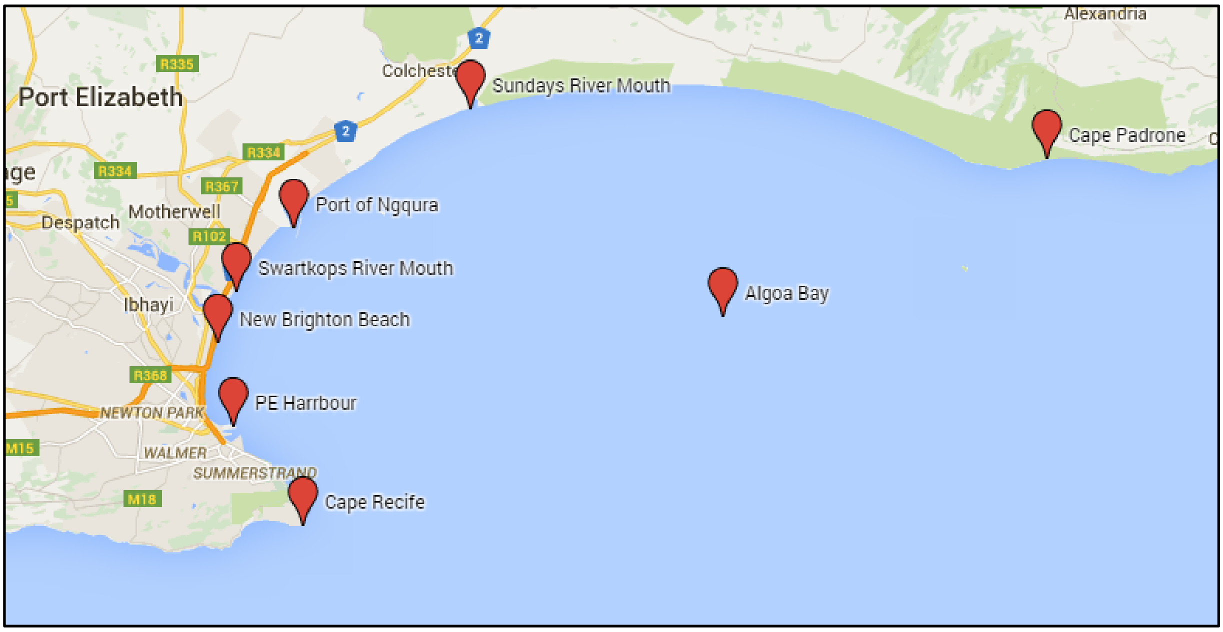


Figure 33: Algoa Bay shoreline sections (Google Maps, 2015).

Starting at the western side of the bay, from Cape Recife headland to the Port of PE, the area can be characterised as mainly a narrow sandy upper beach area underlain by rock. There are also two wider sandy beach areas within this section, namely Pollock and King's Beach respectively (Theron, 2014).

The second section extends from the Port of PE to the New Brighton Beach area. Because of the erosive impact of the PE harbour on the coastline, the necessary protection measures had to be taken. This was done by making use of revetment and dolos armouring, making the upper part of the coastal profile more resistant to erosion (Theron, 2014). The protection measures taken can be seen in Figure 34 below.

Figure 34: Protective measures taken in Algoa Bay (Theron, 2014).

A 400 m short and narrow unprotected shoreline follows the protected section, continuing with the hardened shoreline located at the John Tallant Road foreshore area to the north, located just before the Aloes Jetty and groyne (a coastal structure used to protect against the erosive force of ocean). From there, a 1.8 km stretch of wider sandy shoreline stretches to the mouth of the Swartkops river (Theron, 2014).

A 7.6 km sandy shoreline between the Swartkops river mouth and the Port of Ngqura is a sandy upper beach zone that widens towards the east. This is followed by 18 km-long shoreline from the Port of Ngqura to the Sundays river mouth (Theron, 2014). This section is mostly sandy, but cobbles can be found along the high tide line, especially towards the western side. In the first 9 km from the port, rocky areas are found around the low tide line (see Figure 35 below), followed by an increasingly sandy beach up to the Sundays river mouth with a substantial dunefield widening towards the river mouth (Theron, 2014).

Figure 35: Shoreline from the high tide berm on the western side of the Port of Ngqura (TNPA, 2012).

The last section of Algoa Bay, between the Sundays river and Cape Padrone headland, is where the Alexandria dunefield is located. This stretch is classified as a sandy shoreline (Theron, 2014).

6.2 Sediment characteristics

The CSIR did a sediment quality assessment study to prepare a shoreline assessment specialist study and a dredging and dredge disposal modelling study for the Port of Ngqura before construction began in 2002. Within the proposed development footprint of the Port of Ngqura, surface sediment was collected at four positions (CSIR, 2013a).

The sediment at three of the four stations was dominated by sand (>98% contribution), with the remaining station also mainly sand but with a higher gravel volume (13.39%). Two of the four stations were dominated by medium-grained sand and at the other two stations fine-grained sand was the dominant class. From a textural perspective, three of the stations' sediment were classified as sand and the remaining station was classified as gravelly sand (Theron, 2014). From this it can be concluded that the main sediment component at the Port of Ngqura is fine to medium-grained sand.

6.3 Sediment dynamics

The sediment in Algoa Bay is supplied by rivers, coastal sand transport (longshore transport and headland-bypass dunefields), sand sink (coastal dunefields) and the biogenic production of calcium carbonate (Illenberger, 1992). As mentioned in the previous section, the Sundays and Swartkops rivers are the main rivers draining into Algoa Bay, making them the main suppliers of river sediment to Algoa Bay and its beaches. In this section a more in-depth description of the process of entrainment, transport, and deposition of each sediment class is provided.

According to Illenberger (1992), roughly half of the longshore transport of Algoa Bay originates from the coast west of Cape Recife via longshore transport; a quarter is supplied by the Sundays and Swartkops rivers, and the remainder is supplied by biogenic calcium carbonate produced in the coastal zone.

The sediment supplied to Algoa Bay are found in the form of mud, sand, pebbles, cobbles and even small boulders, but will be divided into three classes for the purposes of this discussion. These are mud, sand and cobbles; where the cobble class will represent pebbles, cobbles and small boulders. Each of these three classes are discussed separately in the next section.

6.3.1 Mud movement

The Swartkops and Sundays rivers are the main suppliers of mud to Algoa Bay. Of these two, the Sundays river supplies a much higher volume because of its sediment load consisting of almost 90% mud. The Swartkops river is estimated to supply approximately 12% of the total mud volume for Algoa Bay (Illenberger, 1992). During river floods, when most sediment in a river is a transported, the mud is carried through the estuaries and surfzone and deposited in the deeper still water of Algoa Bay

6.3.2 Sand movement

The sand in Algoa Bay is supplied by rivers discharging into the Bay, longshore drift around Cape Recife and headland bypass; with both longshore transport and the coastal dunefields being the main suppliers of sand. With the longshore transport rate mainly consisting of sand (as stated in the previous section) it will represent the sand dynamics for Algoa Bay.

The sediment dynamics of Algoa Bay have been a topic under investigation for several years, with numerous studies being done at different locations in the Bay. This rate was a critical factor in the design of the Port of Ngqura in order to determine what the annual volume of sediment that will be obstructed by the Port's breakwaters.

At the Port of Ngqura, the sand-bypassing system was constructed in order to return continuity to the longshore transport (Transnet, 2016). Prestedge Retief Dresner Wijnberg (PRDW) were appointed by Transnet National Ports Authority as lead consultants for the construction of the sand pumping facility. It was estimated that if the system achieves a bypassing rate of 200 000 m^3 per year, the impact of the breakwaters will be kept at a minimum, meaning that the net average longshore transport rate at the Port of Ngqura is 200 000 m3 per year. But since the system started operating, an estimated 1 million m3 of sand accreted on the up-drift section (Rutherford, 2015).

With much speculation as to whether or not the rate was a true representation of the net longshore transport rate at the Port of Ngqura, more research was conducted. Rutherford (2015) did a study on the shoreline changes and longshore transport at the Port of Ngqura and concluded that the net transport rate can be estimated at 160 000 to 200 000 m^3 per year in the northern direction.

6.3.3 Cobble movement

Illenberger (1992) noted that the eastern side of Cape Recife alternates between sandy stretches, like King's Beach, and rocky stretches. There are cobbles along the rocky stretches, well exposed in small patches after easterly storms. However, only small cobbles occur on King's Beach, along the intertidal zone (field observations, 2016), so it seems that the rocky stretches prevent the movement of large cobbles off Cape Recife and along the rocky southern portion of Algoa Bay is mostly, if not totally, restricted to small cobbles. Hence the quartzite cobbles on the beach north of the Swartkops river mouth are mostly derived from the Swartkops river (Illenberger, 1992).

The maximum size and percentage of river-derived cobbles (quartzite) on the beach berms (raised ridge of cobbles or sand found at high tide or storm tide) decrease from the Swartkops river mouth north-eastward (Illenberger, 1992). This characteristic is interpreted to indicate that wave action gradually removes some of these cobbles from the shore and deposits them on the storm berm as longshore drift moves the cobbles north-eastward. A field observation indicates that some calcarenite cobbles, derived from local beach rock, are also contained in the system (Illenberger, 1992).

Through visual inspection the Papenkuils river can be disregarded as a source of coarse material for the area where the Port of Ngqura is located due to the fact that a weir and several other obstructions prevent the movement of coarse material beyond their locations. It is assumed that if some of the coarse material do manage to end up in the coastal system, the dolosse along that shoreline section (stated previously) and the New Brighton pier should prevent further transport alongshore.

The other significant source of cobbles is the Sundays river. According to Illenberger (1992) the maximum size and percentage of river-derived cobbles (quartzite, mudstone and hornfels) on the beach berms east of the river mouth are much higher than west of the mouth, and both decrease from the river mouth eastward (see Figure 7 above). This is interpreted to indicate that these cobbles are gradually removed from the shore and deposited on the storm berm as longshore drift moves the cobbles eastwards. The proportion of calcarenite cobbles increases simultaneously; at Woody Cape (40 km from the river mouth) only a few small river cobbles are left, and east of the Woody Cape cliffs (about 50 km from the river mouth) there are no more river cobbles: the cobble zones here consist solely of calcarenite cobbles eroded from the Woody Cape cliffs (field observations). One may assume that the small river-derived cobbles are gradually deposited in the subtidal zone, as this size of cobbles moves much faster than the large cobbles. These cobbles are therefore carried a little further along the Algoa Bay shore before being deposited (Illenberger, 1992).

It therefore appears that Algoa Bay is a closed system for large as well as small cobbles. The large cobbles are derived mainly from the Swartkops and Sundays rivers and partly from local calcarenite outcrops. The large cobbles move along the beach to the north and east until they are deposited on the storm berm. Small cobbles are more mobile, and are able to move along the rocky southern shore of Algoa Bay; river-derived small cobbles also extend slightly further eastward than large cobbles along the northern Algoa Bay shore (Illenberger, 1992). The lack of a build-up of river cobbles west of Woody Cape implies that most of these cobbles are deposited on the storm berm, spring-tide berm and sub-tidally before they get to Woody Cape. Figure 36 below illustrates the cobble dynamics in Algoa Bay as well as the contribution of the two main rivers.

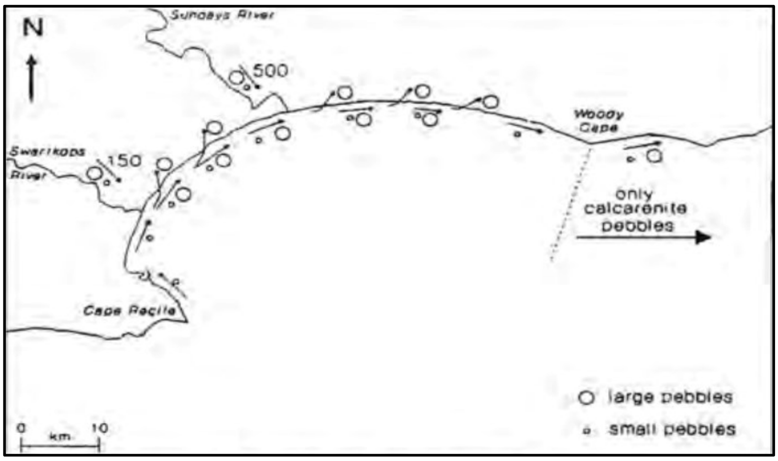


Figure 36: Cobble movement in Algoa Bay (Illenberger, 1992).

6.3.4 Changes in sediment dynamics after the construction of the Port of Ngqura

Illenberger's study was completed in the year 1992, which was prior to the construction of the Port of Ngqura. This means that the effect of the new Port was not taken into account when determining the sediment dynamics of Algoa Bay.

The major impact on the sediment dynamics was the two breakwaters extending seawards (see Figure 1 above), which serves as an obstruction in both directions. With the net longshore transport rate in the north-eastern direction, the sand-bypassing system was installed to only move sediment from the western to the eastern side, resulting in sediment flow in one direction at the Port. This means that transport rates in the western direction are now excluded.

The additional adaption of the sediment dynamics is the origin of the sediment. Because flow is in one direction only, all sediment found at the Port will originate from the western coastline. The coarse material found in the sandtrap will therefore originate from the Swartkops river only, and not both the Sundays and Swartkops rivers, as previously stated by Illenberger.

6.4 Site characteristics

6.4.1 Tidal range

Due to the close proximity of PE to the Port of Ngqura, tidal information for PE has been used to determine the tidal conditions that can be anticipated at the Port of Ngqura. The tidal levels relative to CD are shown in

Table 4 below.

Table 4: Tidal levels at the Port of Ngqura (South African Navy hydrographic office, n.d).

TIDE LEVELS FOR PORT ELIZABETH	Tide Level (m, CD)
Lowest astronomical tide (LAT)	0.00
Mean low water springs (MLWS)	+ 0.21
Mean low water neaps (MLWN)	+ 0.79
Mean level (ML)	+ 1.04
Mean high water neaps (MHWN)	+ 1.29
Mean high water springs (MHWS)	+ 1.86
Highest astronomical tide (HAT)	+ 2.12

Table 4 above indicates that CD coincide with LAT. LAT is the height of the water at the lowest possible theoretical tide. It should also be noted that MSL and mean level (ML) are not the same. MSL is at land levelling datum, 1.026 m above CD, while ML is 1.04 m above CD.

6.4.2 Wave conditions

In general, the two deep-sea locations show a strong predominance of waves (including high storm waves) from the south-westerly quadrant, with a small occurrence of low waves from the east-north-easterly to easterly sector (Theron, 2014).

The nearshore wave conditions were compiled from the offshore data as well as from the Datawell Waverider wave measuring device, located 3 km off the tip of the western breakwater at the Port of Ngqura. The data was analysed for the period from March 2011 to February 2013 with the results summarised in Table 5 below (Theron, 2014).

Table 5: Datawell waverider nearshore wave characteristics (Theron, 2014).

Recorder location	Annual $H_{mo50\%}$ (m)	Annual $H_{mo1\%}$ (m)	Wave period range T_p (s)	Dominant wave direction
Port of Ngqura	1.2	3	4 – 18	SSE

In Section 5.7 the wave conditions at the sand-bypassing at the Port of Ngqura were presented with a wave rose in order to determine the depth of closure. These results represented the wave conditions in the breaker zone at the sand-bypassing system.

6.4.3 Wind conditions

The wind conditions in Algoa Bay are recorded at four locations over different periods. Considering the wind conditions from Cape Recife (2 years of data), Port Elizabeth (23 years of data), Sundays River (9 years of data) and Bird Island (5 years of data), the four main wind conditions, south-westerly, north-westerly, north-easterly and south-easterly can be summarised as follows (PRDW et al., 1997).

The wind regime is dominated by a moderate to strong westerly to south-westerly winds that blow all year. These winds are associated with the passage of east-moving cyclones that develop in the circumpolar westerlies (Tinley, 1985). The wind direction tends to be more westerly in winter and more southerly in summer. The region also experiences moderate to strong easterly winds during spring and summer and weaker northern winds during autumn and winter.

6.4.4 Cross-section

6.4.4.1 Cross-section characteristics

Prior to the construction of the Port of Ngqura, beach monitoring surveys were completed in order to establish the pre-construction profile above 0 m MSL. Figure 37 and Figure 38 display the profiles established by PRDW *et al*. (1997) and Gibb Consulting (1999). Both profiles were taken in close proximity to the bypass system.

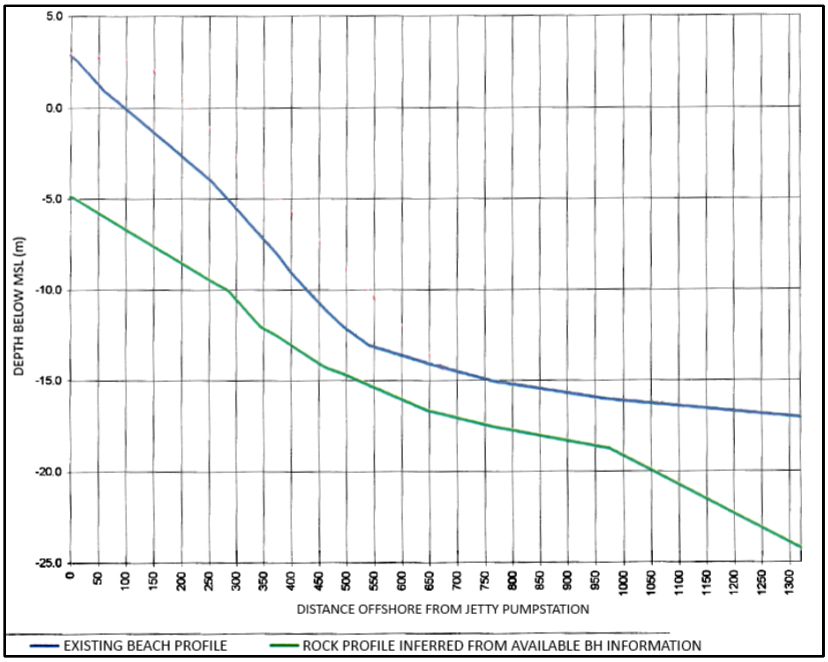


Figure 37: Cross-shore profile (PRDW et al.*, 1997).*

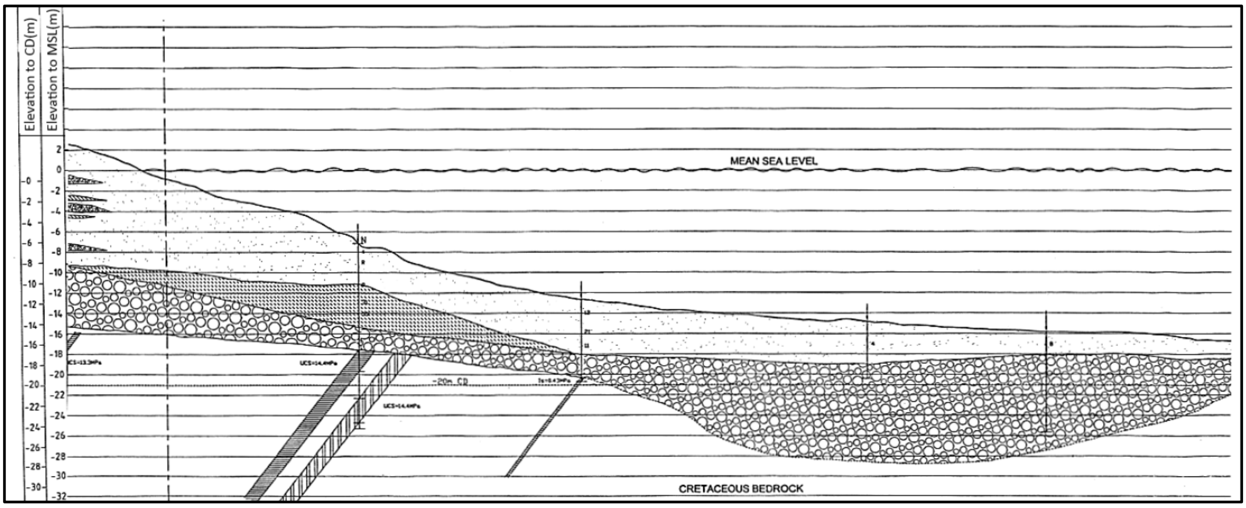


Figure 38: Cross-shore profile by Gibb consulting (PRDW et al.*, 1997).*

Figure 37 and Figure 38 clearly indicate that there is a large volume of sand covering the underlying bedrock. Figure 38 also shows that there is a large volume of coarser sediment located in-between the sand and bedrock.

6.4.4.2 Cross-section after construction

The Port of Ngqura was built in the mouth of the Coega river. Before any construction or designs commenced, the necessary bathymetry measurements were taken at the site using beach-monitoring surveys. It was mentioned in the previous section that the breakwaters of the port will have a distinct impact on the shoreline around the port. Illenberger (2008) did a brief shoreline impact assessment of the Port of Ngqura on the surrounding coastline. The locations where the profiles were measured are shown in Figure 39 below. The only measurement relevant to this study is the profile changes on the up-drift section of the Port (profiles S1-S5).

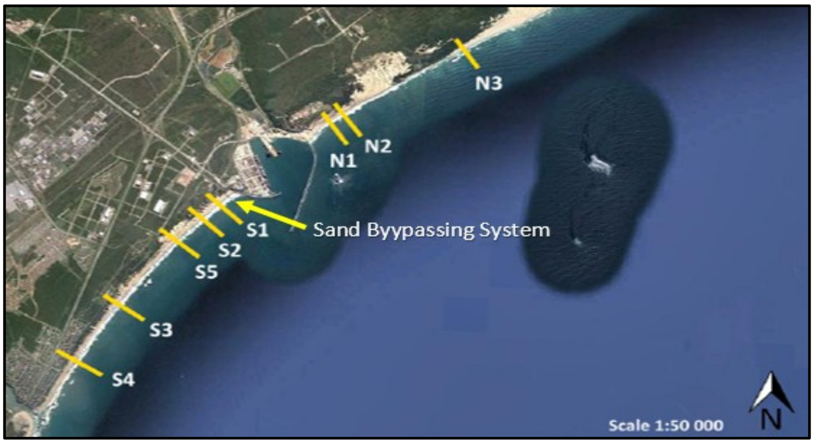


Figure 39: Profile measurement sites (Google Earth, 2015).

With regards to the southern beaches, it was concluded that profile S1 (410 m south of western breakwater) has accreted 80 m in response to the construction of the port, whilst S2 (1180 m south of western breakwater) has accreted 30 m. The rest of the profiles (S3-S5) show no obvious change in profile shape (Illenberger, 2008).

Rutherford (2015) used Illenberger's findings to determine the cross-shore profile after construction in order to ascertain the depth of closure for the study area. The post-construction cross-shore profile at three locations south of the Port is illustrated in Figure 40.

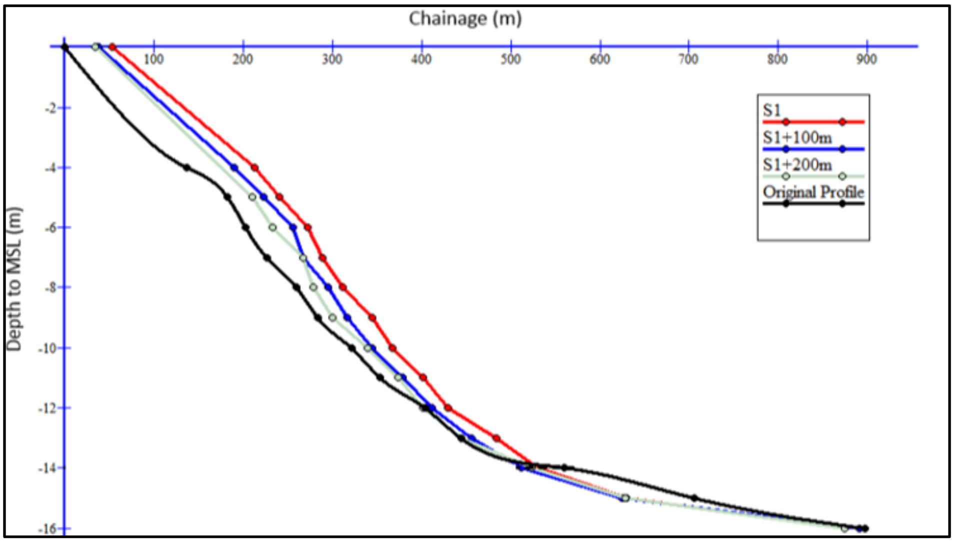


Figure 40: Southern cross-sectional profile after construction (Rutherford, 2015).

The profile is limited to a depth of 16 m below MSL because it was assumed that this was the depth of closure for sand prior to construction of the Port. However, the depth of closure for sand was determined as 14 m in Section 5.7. The average slope at location S1 was determined as 2.5 degrees.

6.4.4.3 Cross-sectional distribution rates

In Section 5.6 it was stated that different transport rates occur along the depth contours of the cross-shore profile. Figure 41 below illustrates the gross-sediment transport at different depths of the sand-bypassing system according to PRDW (1999).

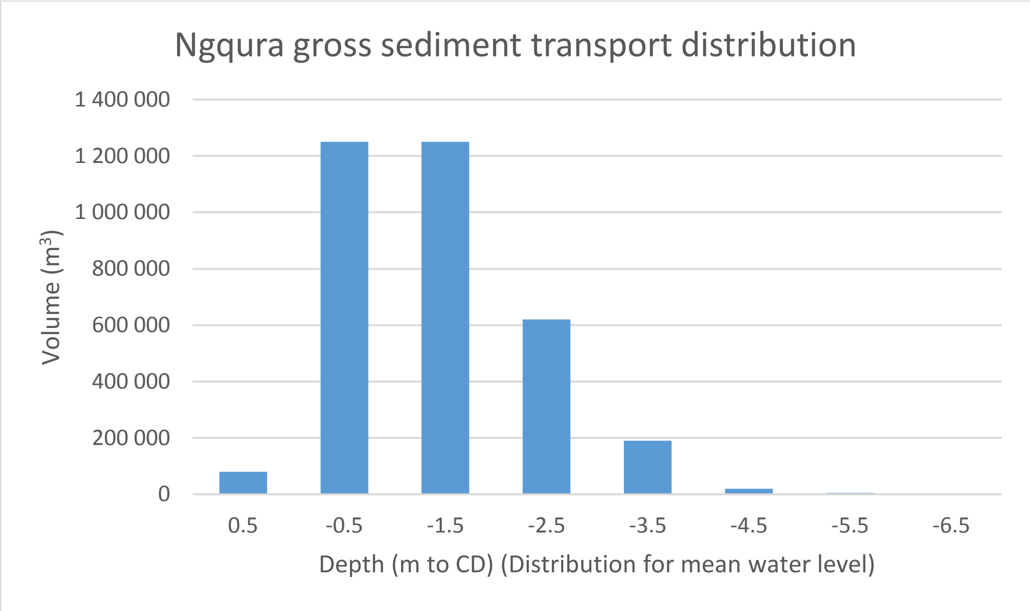


Figure 41: Gross sediment transport along depth profile.

Figure 41 clearly shows that there are two peaks in the gross sediment transport rate at -0.5 and -1.5 m to CD, which implies that this is the section where the highest volume of sediment will transport in, this is the peak section that was referred to in Section 5.7. Figure 41 also show that the majority of the sediment is transported above - 4.0 m to CD or -5.026 m to MSL.

6.4.4.4 Cross-section sediment characteristics

Illenberger (1992) conducted a study on the cross-sectional structure of the shoreline on the coastline just east of the Sundays river. The Sundays river mouth is located on the eastern side of the Port of Ngqura, resulting in higher exposure from the south-westerly swell. This means that higher wave heights are expected at this location, and therefore the heights on the profile should be slightly reduced to provide an accurate representation at the Port of Ngqura. The cross-sectional characteristics of the shoreline with reference to the cobbles size and distribution is illustrated in Figure 42 below.

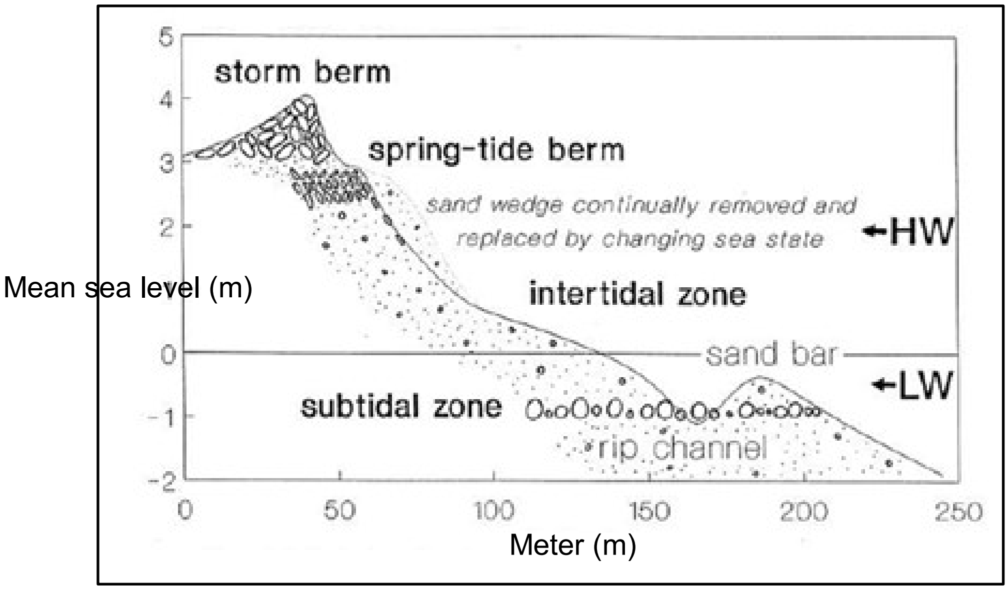


Figure 42: Cross-sectional characteristics (Illenberger, 1992).

The following general portrayal of the cross-sectional structure of the shoreline can be assumed, with reference to cobble size according to Illenberger (1992):

- The storm berm, which is located at the rear end of the beach, has a crest height of +4 m above MSL. The berm consists of the largest cobbles in the profile, with a maximum size of 256 mm. The mean cobble size is 83 mm with a distinctly flat shape. The storm berm can occasionally be covered in sand, and is not usually altered.
- The spring tide berm is slightly lower than the storm berm and located more seawards, with a crest height of +3 m above MSL. The berm consists of medium-sized cobbles with a mean size of 53 mm. The spring tide berm is usually altered during a spring tide or a storm.
- The section ±1.5 m to MSL in the figure consists of cobbles with a mean size of 23 mm.
- The section -0.5 m to -1 m MSL consists of cobbles with a mean size of 86 mm, which is slightly larger than the spring tide berm.

The locations of the different size of coarse material from Figure 42 also provide a rough estimate of the main transport zone for cobbles in the longshore transport. The location of the cobbles on the storm and spring tide berm can be ignored because it is an occasional occurrence. Therefore, the main transport zone is between +1.5 and -1.5 m to MSL or +2.53 and -0.47 m to CD. This is a conservative approximation due to the higher wave exposure at the Sundays river mouth.

6.5 Conclusion

This chapter focused on different site aspects of Algoa Bay. The first section described the shoreline characteristics of Algoa, which provided the basis for the sediment dynamics in Algoa Bay. Research from Illenberger (1992) revealed that the Swartkops and Sunday river were the main sources of coarse material for Algoa Bay. After the construction of the breakwaters of the Port, the sediment dynamics changed, which resulted in the Swartkops river becoming the main source of coarse material found in the sandtrap.

The Papenkuils river was disregarded as a source of coarse material due to the installation of weir and several other obstructions, which prevented the movement of coarse material beyond their locations. The information also revealed that some of the coarse material were derived from local beach rock that is also contained in the system.

The site characteristics provided the tidal range, wave conditions, wind conditions and cross-section characteristic at the Port of Ngqura. These aspects will provide key elements for the design of the conceptual solutions in Chapter 7.

Chapter 7: Conceptual solutions

In this section, the knowledge obtained from the coastal system at the Port of Ngqura will be used to produce conceptual solutions to prevent the obstruction of the jet pump intakes of the sand-bypassing system. Each solution will focus on certain aspects or properties in order to support the design. The designs of the conceptual solutions were drawn in Autodesk®Autocad® (2016).

7.1 River abstraction

7.1.1 Introduction

The premise on which this proposed solution is based, is the fact that before the construction of the Port of Ngqura, the two main sources of coarse material for Algoa Bay was the Swartkops and Sundays river (as stated in Section 6.3). Once the construction of the Port was completed, the sediment dynamics changed, restricting flow in one direction. This process resulted in the Swartkops river becoming the largest source of coarse material in the western section of Algoa Bay. If the coarse material supply from the Swartkops river can be restricted or managed, the volume of coarse material that will be found in the sandtrap will reduce drastically. This will be done by abstracting the coarse material from the Swartkops river by using an excavation. The details of the excavation will be explained in the following section. The yearly sediment yield from the Swartkops river according to Illenberger (1992) is depicted in Table 6 below.

Table 6: Swartkops river sediment yield.

	Coarse material (m^3)	Sand (m^3)	Mud (m^3)
Total volume to sea	150	25 000	51 000

In Chapter 3 it was found that particles with a diameter larger than 150 mm can potentially obstruct the intakes of the jet pumps. The coarse material group in Table 6 represents all particles larger than 20 mm, which means that this entire volume of 150 m^3 cannot be considered as potential problems for the sand-bypassing system. But due to the lack of information regarding particles larger than 150 mm, a conservative approach is followed, by assuming that the entire volume consists of particles equal to, and larger than 150 mm. This value will be used throughout the chapter for all the conceptual solutions.

In Chapter 4 it was stated that the coarse material located in the sandtrap at the Port of Ngqura originates from the rock revetment, remnants of the temporary construction works, and natural sources. This means that the river abstraction in the Swartkops river will only reduce the coarse material supplied by the natural source and not from the temporary construction works and the revetment. The additional adjustments required at the rock revetment and the remnants of the temporary construction works will be discussed in Chapter 8.

As stated above, the natural source will only be reduced and not completely mitigated due to the coarse material derived from local beach rock and coral. These two sources will continue to migrate into the sandtrap. Although this is a small fraction compared to the Swartkops river, as stated in Section 6.3, this process cannot be prevented by this conceptual solution. Therefore, occasional clearance of the sandtrap will still be required.

7.1.2 Location

The location of the abstraction point according to Basson (2016) must be outside the boundary of the estuary because of the restrictions placed on construction within an estuary. As Basson explains, any excavation in the estuary will alter the water levels, which will have a major impact on the surrounding environment (for example, a rise in water level will create safety issues for the settlement located on the bank of the estuary).

The location must therefore be upwards of the tidal limit (which is the farthest point upstream on the Swartkops river at which a tidal variation in water level is observed (Bureau of Meteorology, 2016)). The layout of the Swartkops estuary and river is illustrated in Figure 43 below.

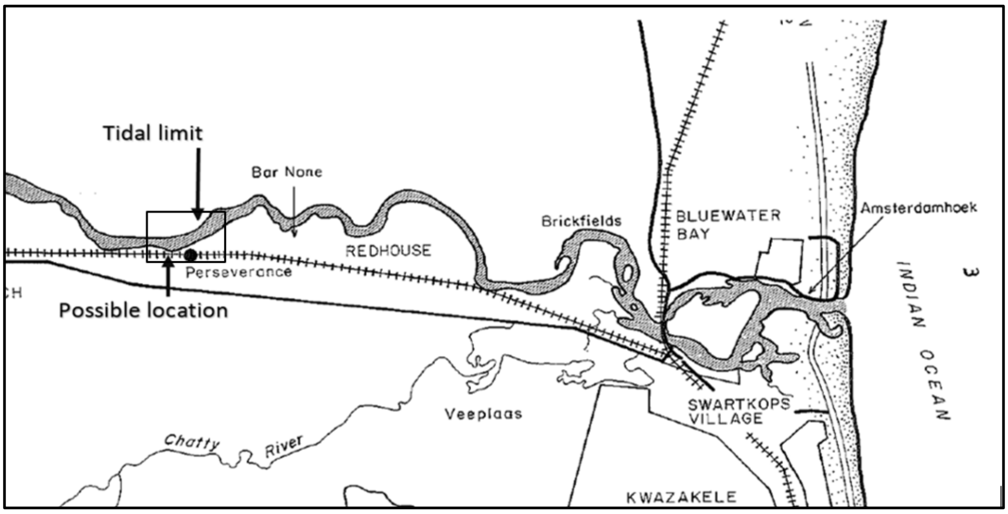


Figure 43: Swartkops estuary (Baird et al.*, 1986).*

Figure 43 above also displays the location of the tidal limit for the Swartkops estuary, which is located upstream from the town Perseverance. A possible location for the abstraction works is also depicted in this figure.

Because the location of the excavation is so far upstream in the Swartkops river, it is critical to understand that there will still be coarse material located below the extraction point and in the western section of Algoa Bay. This source will decrease with time, as the coarse material will either end up on the storm berm or in the sandtrap. This means that the solution will not have a direct impact on the coarse material supplied by the natural source, but in the long term it will reduce drastically.

7.1.3 Design

The design of the excavation will only be conceptual, due to the lack of information available about the cross-sectional profile of the Swartkops river at the chosen location in Figure 43. The key factors that will determine the parameters of the design are discussed in this section.

7.1.3.1 Depth of excavation

The fundamental principle that this concept is based on is the continuity of flow for open channel flow, which states that if the flow is constant in a channel, the product of the area and velocity will be the same for any two cross sections within that channel (Chanson, 2004). The continuity of flow is represented by Equation 11 below.

Equation 11: Continuity Equation

$$Q = VA$$

- Q = the volumetric flow rate (m^3/s)
- V = the mean velocity (m/s)
- A = cross sectional area of flow (m^2)

This means that if the depth of the channel is increased, which results in an increase in the area of that cross-section, it will result in an immediate decrease in velocity. By applying this concept, the velocity decreases to a specific point where it is lower than the boundary condition for incipient motion for cobbles with a diameter larger than 150 mm, causing them to settle out.

The particles smaller than 150 mm will stay in motion because of the smaller velocities required to keep them in motion, as stated in Section 5.4. However, while in theory all of the particles smaller than 150 mm will continue to move past the excavation, in reality some of the smaller particles will still deposit in the enlarged area.

7.1.3.2 Flow conditions

The flow rate is one of the required parameters in Equation 11. The most important factor in selecting a flow rate is that particles with a diameter of 150 mm are not likely to be transported during normal flow conditions as described in Section 5.4, but rather during river floods when the velocities are much higher. The abstraction must therefore be designed for river flood conditions.

In Chapter 5 it was explained that smaller particles require a lower velocity to set and keep the particles in motion, whereas a larger particle requires a larger velocity. A high return period flood will result in higher flow rates, which will require a deeper excavation (larger area). The problem is that with a deeper excavation, the flow velocity during normal condition will also decrease drastically; causing the smaller particles to deposit in the excavation and requiring frequent clearance. Therefore, a balance must be found between the design flood conditions required to transport large sediment, while also limiting the deposition of smaller particles during normal flow conditions.

In order to determine the flow rate for floods with a specific return period, the flow conditions during floods were required. The data was obtained from the Department of Water and Sanitation for the M1H012 meter station for the period 1994 to 2016. The meter station is located upstream of the potential location, but the assumption is made that the flow rate remains constant over this distance.

The data revealed that 22 floods occurred between 1994 and 2016, these floods were arranged from the largest to the smallest in order to obtain a ranking for each flood. The flow rate with different return periods were calculated using Equation 12 below. The data was plotted and the line of best fit was obtained. The result was used to calculate the return periods for the Swartkops river as shown in Table 7 below.

Equation 12: Return period (Poisson probability distribution)

$$R = \frac{n+1}{m}$$

- R = return period in years
- n = number of floods (22)
- m = rank of specific flood

Table 7: Flood return period for the Swartkops river.

Return period	Flow (m^3/s)
1:1	50
1:3	213
1:5	377
1:10	784
1:50	4048
1:100	8127

8.1.3.3 Length of the excavation

Downstream of the excavation, the Swartkops river will return to the original cross-section, which means that the velocity will increase again. Therefore, the length of the structure will ensure that the velocity is lowered for a long enough period to allow the larger particles to settle out.

The length of the excavation will also provide sufficient volume to store the particles during a storm. A conservative approach is followed by assuming that the volume of coarse material delivered to Algoa Bay will be equal to the volume that is transported past the abstraction point and that the excavation will be cleared annually. Therefore, the volume must be large enough to store the annual volume of 150 m^3 (from Table 6) and also take into account the fraction of the smaller sediment that will end up in the excavation. It is unlikely that mud will deposit in the excavation because during low flows the mud will deposit in the excavation, but it will be brought into resuspension during higher flows. Therefore, the storage volume needs to accommodate for both coarse material and sand.

The length will also determine the regularity of maintenance required at the excavation. If a small length is chosen for the design, the overall volume is lower; resulting in an increase in the regularity for maintenance.

7.1.3.4 Protection measures

The sudden change in depth of the river will cause erosion in the upstream direction of the excavation, which will reduce the performance of the system. This section of the excavation must therefore be protected. According to Basson (2016) the most simplistic and cost-effective method will be to protect the upstream bank with rock-filled gabions. The layout and features of the conceptual design was designed in Autodesk® Autocad® (2016) as shown in Figure 44 below.

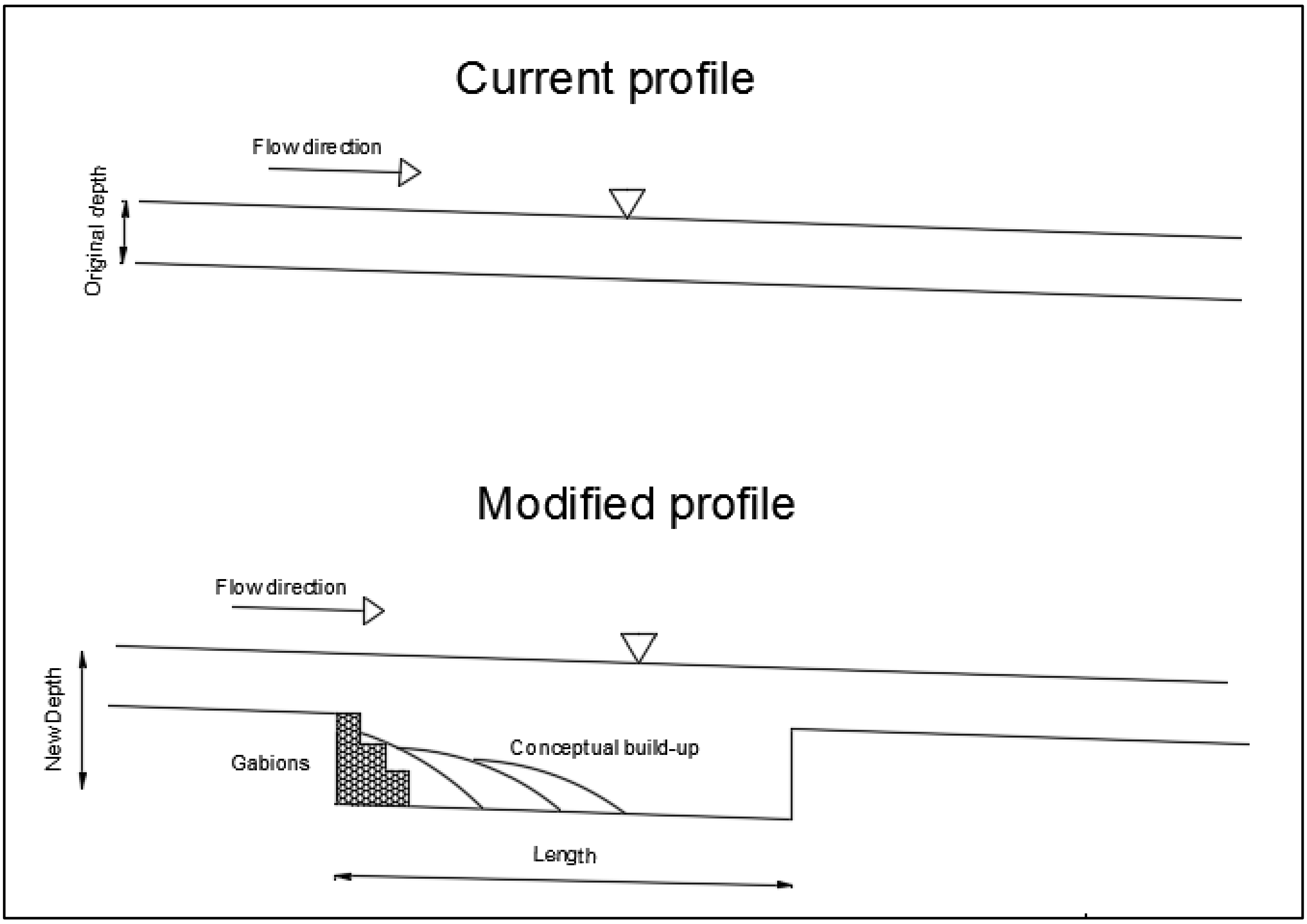


Figure 44: Cross-section of river profile.

7.1.4 General concerns

The challenges pertaining to this proposed solution are as follows:

- environmental impact assessments (EIA) will be required to determine the impact of the project, which is a time-consuming process and the outcome will determine whether or not the project can continue;
- as stated earlier, there will be no immediate effect on performance of the sand-bypassing system, which means that the current prevention methods of the sand-bypassing system, as stated in Section 3.2, will have to continue;
- the coarse material derived from local beach rock and coral will continue to migrate into the sandtrap, which will still require clearance from time to time;
- the excavation will have to be cleared after every storm and during the year because if the depth is not maintained, the excavation will not be effective;
- the material that is cleared from the excavation must be relocated to another location, which will require another EIA study; and
- the excavation will capture both large and smaller particles, causing the excavation to fill up with undesirable material.

7.1.5 Concluding remarks

While, in theory, the river abstraction as a solution is one of the more basic and cost-effective concepts, in reality this is not the case. The major problem with this concept is the fact that Algoa Bay is already experiencing problems with erosion caused by current interruptions in the sediment transport system. Moreover, with the Swartkops river being one of the main sources, extensive EIAs will be required to determine the impact of this design. The EIA process is a very time-consuming process that will only prolong the period until the effect of the solution will be realised, which will pose a major challenge because this is already a long-term solution.

The excavation is also located far away from the sandtrap, which means that the coarse material that is a direct risk (in close proximity) to the sand-bypassing system cannot be obstructed. In other words, some of the coarse material that is supplied by the Swartkops river might end up on the storm berm before it reaches the sand-bypassing system, as stated in Section 6.3. This results in the removal of unnecessary coarse material that is not a direct threat to the sand-bypassing system.

River abstraction has the potential to be a viable option for the Port of Ngqura if time is not of the essence. However, with the impact of the Port of Ngqura on surrounding shoreline of Algoa Bay only increasing and the required bypassing rates not being achieved, this concept might arguably not be the best solution in the current state.

7.2 Submerged groyne

7.2.1 Introduction

The premise that the submerged groyne solution is based on is the fact that particles with a diameter of 150 mm and larger will theoretically be transported in a bedload manner, whereas sand particles being the majority of the longshore transport will travel in suspension. If a structure can serve as a barrier for the larger particles but remains low enough to allow most of the suspended particles to pass by, the concept can be used as a potential solution.

A structure with these properties can be found in the form of a submerged groyne. Groynes are hydraulic structures constructed with the main purpose of stabilising a stretch of beach against erosion that is caused primarily to a net longshore loss of beach material (Coastal Processes, Hazards, and Society, n.d.). This prominent feature of a groyne can be used to stabilise (obstruct) coarse material in the longshore transport.

The proposed groyne will be constructed perpendicular to the pre-project shoreline; with its orientation, length, height and permeability determined in the following section. A groyne can consist of several types of material, but a rubble mound groyne is opted for because it:

- has a relatively low in cost;
- requires a basic construction method;
- has the ability to withstand severe wave loads;
- has a long life expectancy;
- demands minimal maintenance; and
- its permeability can be adjusted to requirements (Abdelhamid, 2013).

The submerged groyne will only prevent the coarse material supplied by natural sources from migrating into the sandtrap. As stated in Chapter 4, the rock revetment and the remnants of the temporary works will still serve as a source of coarse material. Therefore, the additional adjustment required to for these two sources will be explained Chapter 8.

7.2.2 Location

In section 6.3, it was stated that the net longshore transport at Algoa Bay is primarily in the north-eastern direction. To therefore prevent the migration of the coarse material into the sandtrap, the groyne must be located to the west of the sand-bypassing system.

Figure 45 below illustrates the shoreline evolution south of the breakwater for the period 1978 to 2014. The ideal location of the groyne is in the section where the shoreline remained rather constant after the construction of the Port to decrease the impact of the groyne on the sand transport towards the sandtrap. A possible location is indicated in Figure 45 below, which is located 800 m from the sand-bypassing system in the western direction.

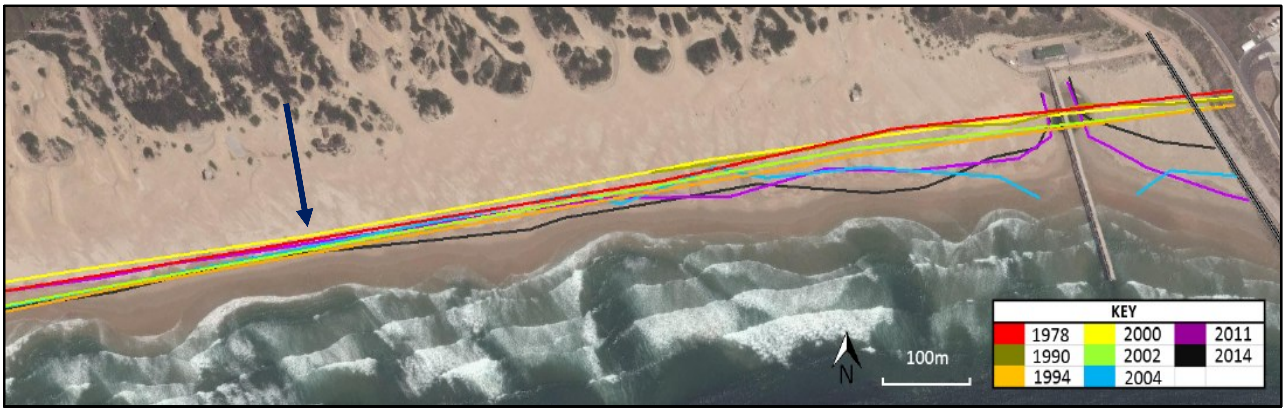


Figure 45: Possible location for the groyne (Rutherford, 2015).

7.2.3 Design

The first modification from a normal groyne will be that one-size armour unit will be used throughout the entire groyne; which will make the construction simpler. This will also avoid transition areas (the section where the diameter of armour rock used changes) that may create vulnerable sections in the groyne.

The conditions at the head of the groyne (the deepest section of the groyne) will serve as the design conditions for the entire groyne. The reason for following this approach include that a depth-limited state is assumed, which also means that the highest wave conditions will occur at the deepest section. This will result in a conservative design approach for the groyne because the entire groyne will be designed for the extreme conditions at the head of the groyne. The design of the groyne will therefore be for a submerged rubble mound groyne because this section will always be below the water level.

The main objective of the core is to avoid undesirable transmission of waves and sediment transport through the structure (Gravesen, 2008), which is not a requirement for the submerged groyne. Therefore, the structure will be constructed without a core, which will increase the permeability of the groyne; allowing some of the smaller sediment (for example sand) to move through the structure (U.S. Army Corps of Engineers, 2006). This will decrease the volume of sand obstructed from the longshore transport by the groyne. The main disadvantage of this approach will be for the construction of the groyne, where the core usually serves as access for construction. The construction method will be discussed Section 7.2.4.

7.2.3.1 Length

The rubble mound groyne will extend up to a depth of 7 m (the depth of closure for cobbles with a diameter of 150 mm), which is located about 230 m offshore from the MSL datum. Although it was stated in Section 6.4 that the majority of the sediment (both fine and coarse) would transport above

-4 m to CD, a conservative approach is followed to obstruct all the coarse material in the longshore transport. Therefore, the head of the groyne will be located at the depth of closure for cobbles.

The landward end of the groyne must extend to a HAT to stay beyond the normal zone of beach movement, which will avoid outflanking by back scouring (U.S. Army Corps of Engineers, 2006). The proposed layout of the groyne is shown in Figure 46 below.

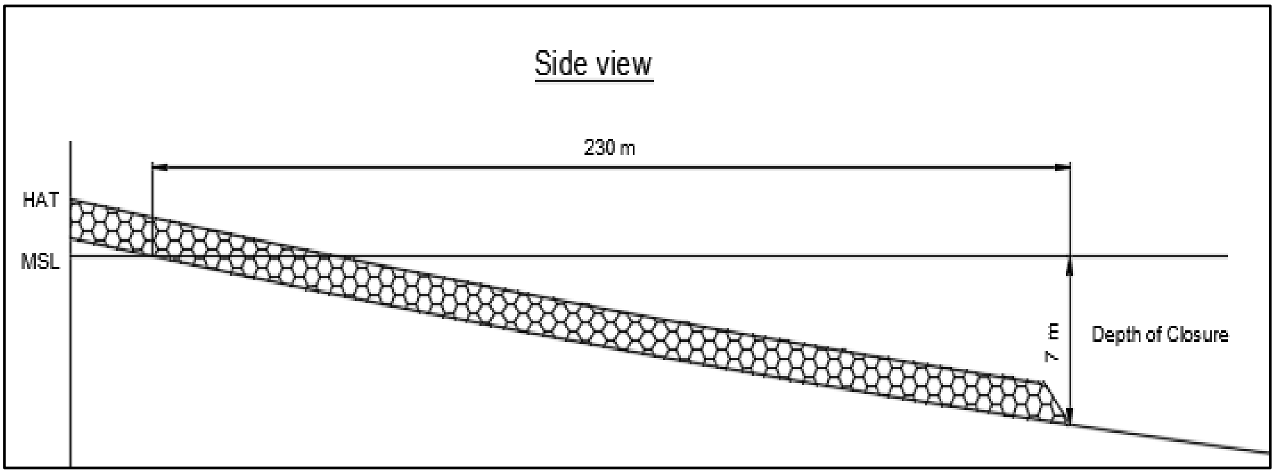


Figure 46: Layout of the groyne.

7.2.3.2 Armour layer

In Section 5.3 it was stated that a bedload particle usually moves within a region of less than 10 to 20 times the particle diameter (Chanson, 2004). It is assumed that a lower limit of 10 is used. Taking this into account, it can be deduced that the minimum height of the groyne must be equal to 10 times the particle diameter of 150 mm, which is equal to a height of 1.5 m.

As previously stated, a depth-limited state is assumed, which is the relationship between the largest wave height that is that can occur in a certain depth (the design depth). The groyne will be designed for breaking wave conditions at the head of the groyne, which means that wave run-up and wave setup can be excluded from the design depth of the groyne.

Design depth

The water level fluctuations are affected by the following aspects according to the Shore Protection Manual (CERC, 1984):

- tsunamis;
- seiches (standing wave in an enclosed or partially enclosed body of water)
- wave setup;
- storm surges;
- astronomical tides;
- climatological variations; and
- secular variations.

In the South African context, fluctuations like long period waves (e.g. edge waves) and tsunamis are only of secondary importance (Wijnberg, 1993) and with the sand-bypassing system not located in an enclosed basin, it can be assumed that seiches will not affect the water levels at the sand-bypassing system.

Astronomical tides

It is assumed that the input water level is set at mean high water springs (MHWS) when the storm surge is determined, because in South Africa spring tides occur every two weeks, which means that the chances of storm waves coinciding with spring high tides are relatively high (Theron, 2016). The MHWS as stated in Section 6.4, is at + 0.83 m to MSL.

Storm surges

Due to the close proximity of PE to the Port of Ngqura, it can be assumed that the extreme residual still-water level estimates for PE, excluding residual tides for different return periods, will be relatively similar for the Port of Ngqura. The still-water level estimates for the Port of Ngqura are shown Table 8 below.

Table 8: Extreme residual still-water level for the Port of Ngqura (Theron, 2016).

Return period in years	**1**	**5**	**10**	**25**	**30**	**40**	**50**	**100**
Residual sea level to MSL (m)	0.38	0.60	0.63	0.66	0.67	0.67	0.64	0.70

Climatological variations

The rise in sea level will determine the future storm surge levels. Theron (2016) determined the sea level rise (SLR) forecasts of 0.15 m, 0.35 m and 1 m by 2030, 2050 and 2100. These results were used to construct the SLR for other periods as shown in Figure 47 below.

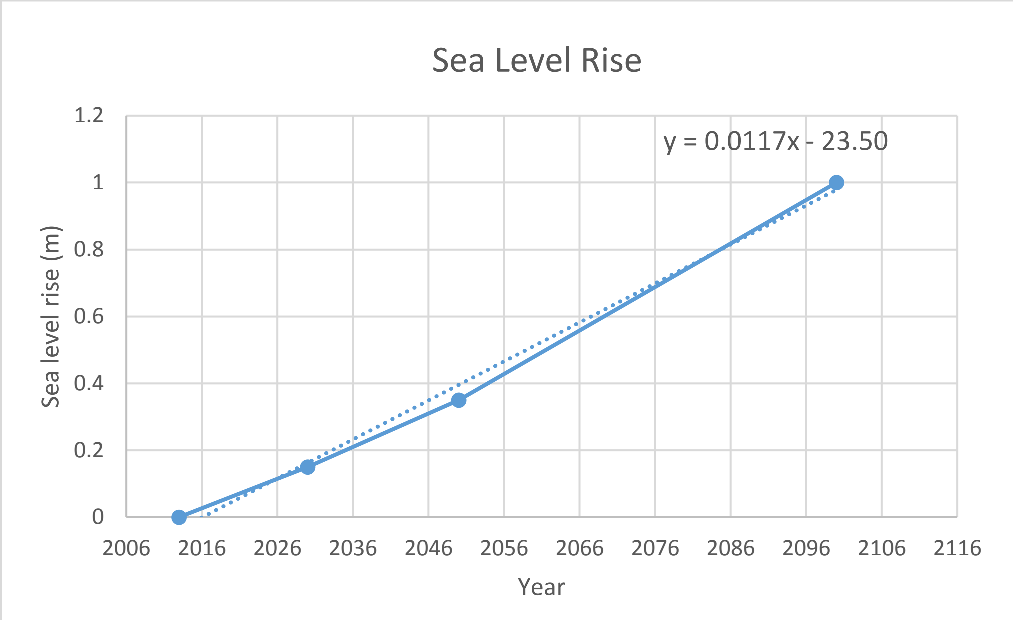


Figure 47: Sea level rise.

The line of best fit was also determined as shown Figure 47 above. The formula of this line was used to determine the SLR for different return periods as shown in Table 9 below.

Table 9: Sea level rise for different return periods.

Return period in years	**1**	**5**	**10**	**25**	**30**	**40**	**50**	**100**
Sea level rise (m)	0.10	0.14	0.20	0.38	0.44	0.55	0.67	1.25

Water level

The total water fluctuation is equal to the astronomical tide plus the storm surge and SLR. The results for the different return periods are shown in Table 10 below.

Table 10: Total water fluctuations for different return periods.

Return period in years	**1**	**5**	**10**	**25**	**30**	**40**	**50**	**100**
Water fluctuation to MSL (m)	1.30	1.58	1.67	1.87	1.94	2.06	2.19	2.79

For the conventional design of a coastal structure a 1:100 year return period will be used Sorensen (2005). However, the groyne will only effect the performance of the bypassing system and will have no impact on human safety. Therefore, a 1:50 year return period can be used according to Sorensen (2005). This delivers a water fluctuation of 2.2 m (Table 10). The extreme water level plus the depth at the head of the groyne is therefore equal to the depth of closure, which is 7 m to MSL plus the 2.2 m contributed by the water fluctuations. The total depth at the head of the groyne is equal to 9.2 m to MSL. The relationship between the breaking wave height and water depth for a depth-limited sea state is shown in Equation 13 below (CEM, 2006).

Equation 13: Relationship of breaking wave height and the breaking water depth (CEM, 2006)

$$d_b = 1.28H_b$$

- d_b = breaking water depth (m)
- H_b = wave height at breaking (m)

The design wave height is equal to 9.2 m, divided by 1.28, which is equal to 7 m. In order to determine the armour layer diameter, two methods will be used as shown in Equation 14 and Equation 15 below. Both these equations determine the required diameter for a submerged breakwater.

Equation 14: Van der Meer (1991)

$$\frac{h'c}{h} = (2.1 + 0.1S)e^{-0.14N_s^*}$$

- $h'c$ = height of the structure over seabed level(m)
- h = water depth (at the deepest section of the groyne) (m)
- S = relative eroded area
- N_s^* = spectral stability number = $\frac{H_s}{\Delta D_{n50}} s_p{}^{-1/3}$
- H_s = significant wave height (m)
- $\Delta = \frac{\rho_s}{\rho_w} - 1$
- ρ_s= mass density of armour unit (kg/m^3)
- ρ_w= mass density of water (kg/m^3)
- D_{n50}= diameter of armour unit (m)
- s_p = local wave steepness
- $s_p = \frac{2\pi H_s}{g T_p{}^2}$
- g = gravitational acceleration (m/s^2)
- T_p = peak wave period (s)

Equation 15: Burcharth et al. (2006)

$$D_{n50} = \frac{H_s}{1.2\Delta} + 0.36R_c$$

- D_{n50}= diameter of armour unit (m)
- H_s= significant wave height (m)
- $\Delta = \frac{\rho_s}{\rho_w} - 1$
- ρ_s= mass density of armour unit (kg/m^3)
- ρ_w= mass density of water (kg/m^3)
- $R_c = h_c - h$ = freeboard (m)
- h_c= crest height (m)
- h = water depth (m)

The required armour layer diameter according to the van der Meer (1990) and Burcharth *et al.* (2006) methods are displayed Table 11 below.

Table 11: Required armour layer diameter.

Method	**Armour layer diameter(m)**
Van der Meer (1991)	1.2 m
Burcharth (2006)	1.6 m

Table 11 above indicates that these two methods delivered varying results and because a conservative approach was already followed to determine the design wave height, the average of the two methods were taken as the design amour unit diameter. The average for these methods resulted in an armour unit diameter of 1.4 m. The relationship between the mass of a unit and the diameter, according to van der Meer (1990), is shown in Equation 16 below. The assumptions that were made to determine the mass of the units are shown in Appendix A.

Equation 16: Median mass with respect to diameter

$$M_{50} = \rho_s (D_{n50})^3$$

- M_{50} = median mass of a unit (kg)
- ρ_s = mass density of armour unit(kg/m^3)
- D_{n50} = nominal diameter of stone (m)

The design mass of a single armour unit is equal to 7 271 kg. The design amour unit also satisfies the minimum height requirement of 1.5 m, as stated earlier, for a two-layer rubble mound groyne.

7.2.3.3 Under layer and geotextile

According to Schoonees (2015), an under layer unit is between a 10th and a 15th of the armour unit's mass. A conservative approach results in the use of the lower limit. The relationship between the mass of a unit and the diameter was shown in Equation 16 in the previous section. This results in a single under layer unit with a diameter of 0.64 m and a mass of 727 kg.

The traditional method for the design of a breakwater/groyne is to use a core beneath the under layer. As stated earlier, however, the core will not be used to increase the permeability of the groyne.

Below the armour layer a geotextile (i.e. permeable geosynthetic consisting solely of textiles with the ability to separate, filter and protect) will be used to provide the necessary hydraulic and mechanical properties to prevent leaching of the underlying soil. The geotextile is faster and easier to install than stone layers, and its factory-controlled properties result in a consistent performance (Terram, 2015).

7.2.3.4 Crest and base width

The crest and base width according to the U.S Army Corps of Engineers (2006) is displayed in Equation 17 and Equation 18 below.

Equation 17: Crest width (CERC, 1984):

$$B_c = 3K_\Delta D_{n50}$$

- B_c = crest width (m)
- K_Δ= layer coefficient
- D_{n50}= diameter of armour unit (m)

Equation 18: Base width (CERC, 1984):

$$B_b = B_c + 2(h'c)\cot\alpha$$

- B_b = Base width(m)
- $h'c = \text{design crest elevation}$ (m)
- α = slope of groyne (°)

Equation 17 and 18 deliver a crest width of 4.2 m and a base width of 23 m. The assumptions made in order to determine the crest width and height are shown in Appendix A.

7.2.3.5 Toe design

It is assumed that the under layer units will be used to create the toe of the groyne. The toe of the groyne will be designed according to van der Meer (1990), with the height of the toe determined with Equation 19 below.

Equation 19: Van der Meer (1990)

$$\frac{H_s}{\Delta D_{n50}} = 10.5\left(\frac{h_t}{d}\right) - 1.9$$

- H_s= significant wave height (m)
- $\Delta = \frac{\rho_s}{\rho_w} - 1$
- ρ_s= mass density of armour unit (kg/m^3)
- ρ_w= mass density of water (kg/m^3)
- D_{n50}= diameter of armour unit (m)
- h_t = water depth-toe height(m)
- d = water depth (at the deepest section of the groyne(m)

The toe height according to the van der Meer (1990) method delivered a height of 1.45 m, which requires more than two layers of under layer units to reach the toe height. Therefore, three layers will be used to reach the required toe height.

7.2.3.6 Groyne round head

The entire groyne was designed for the round head section. Therefore, no additional design adjustments are required to accommodate the effects at this section. The final layout of the groyne is shown in Figure 48 below.

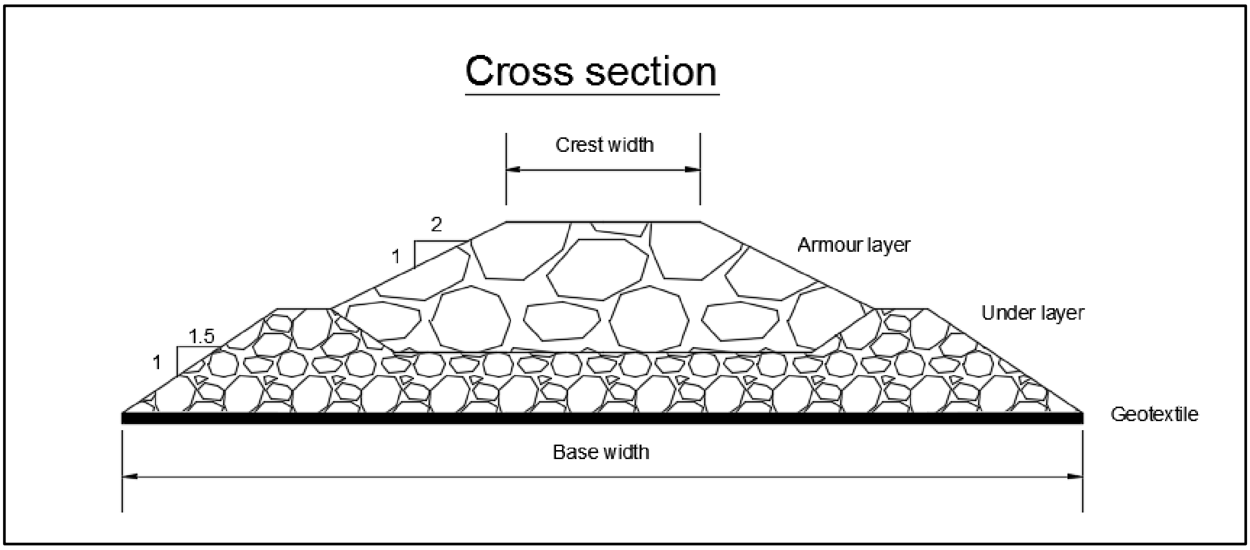


Figure 48: Groyne design.

7.2.4 Installation

In the case of a normal rubble mound breakwater (not submerged), the part of the breakwater that is already completed is used for access to construct the next section, but for the submerged breakwater this is not possible.

In order to construct the majority of the groyne on land, the construction will be performed during LAT. The section that is submerged will have to be completed from barges by placing/dumping the material during calm conditions. The armour layer has a size of 1.4 m with a median mass of 7271 kg, which will be placed in the correct position by a water-based crane.

7.2.5 Capacity

The overall capacity of the structure is directly dependent on the length and shape of the groyne. The simple design of a straight groyne with an offshore length of 230 m was chosen.

The problem with determining the capacity of the groyne is that the groyne will not solely obstruct the yearly volume of 150 m^3 of coarse material in the longshore transport, but also a fraction of the sand in the longshore transport. Therefore, the capacity of the groyne is reached when both sand and coarse material fill up the available capacity.

The CSIR (1998) modelled the longshore transport in 1998 with the objectives of determining the shoreline changes adjacent to the breakwaters using the specified wave conditions, and establishing the accretion against the southern breakwater. If assumed that the same shape will build up against the groyne (because of the same wave conditions) just not to the same height, a relative volume can be calculated as shown in Figure 49 below.

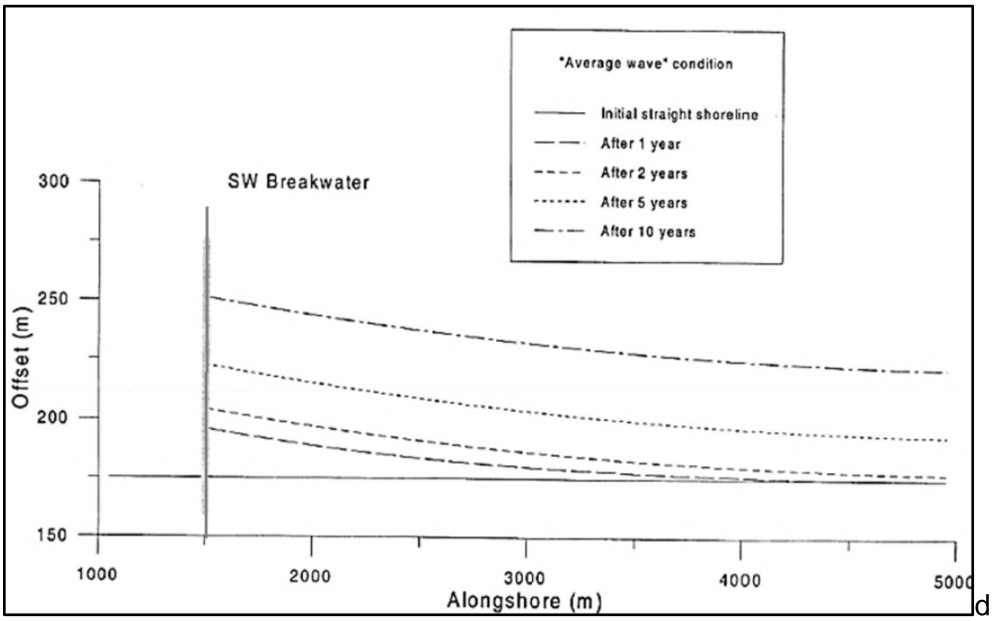


Figure 49: Accretion against the western breakwater (CSIR, 1998).

A relative relationship from Figure 49 above can be determined by the vertical and horizontal accretion length. A 20 m build-up in the vertical direction originates from 2000 m in the down-drift direction, as derived from Figure 49. Using this relationship, the accretion against the groyne was determined as shown in Figure 51 below. The shape represents the maximum capacity of the groyne; if any further accretion occurs, sediment will bypass the groyne. The volume shown in the figure is for a thickness of 1.5 m as this is also assumed to be the maximum build-up height against the groyne.

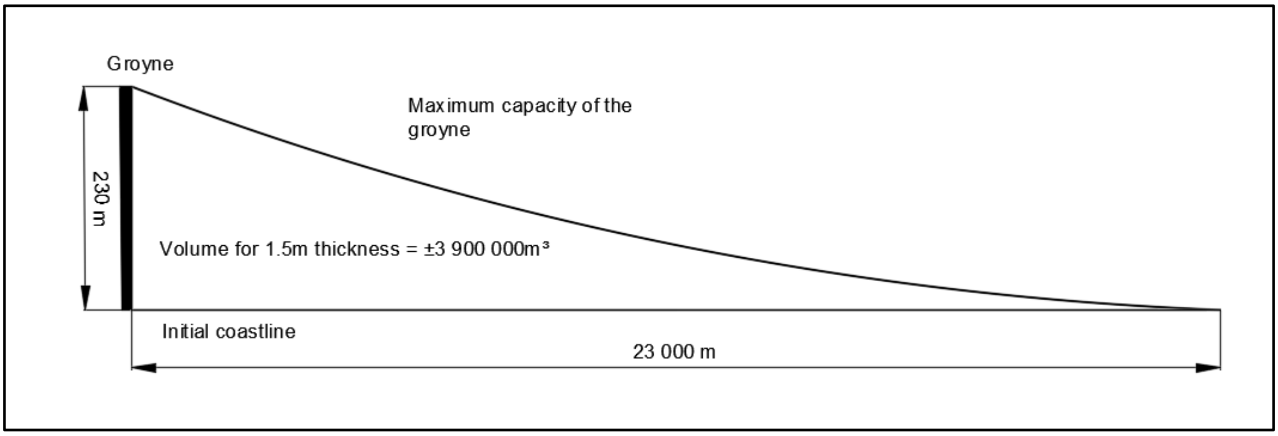


Figure 50: Accretion against the submerged groyne.

It was previously assumed that the thickness would be 1.5 m, which represents a thickness of 1.5 m throughout the shape. This is not, however, the case. The build-up shape against the groyne is assumed to be as displayed in Figure 51 below. If the volume is roughly estimated only to build up to half the estimated thickness, as displayed in Figure 51, this results in a maximum capacity of 1 950 000 m^3 for both sand and coarse material.

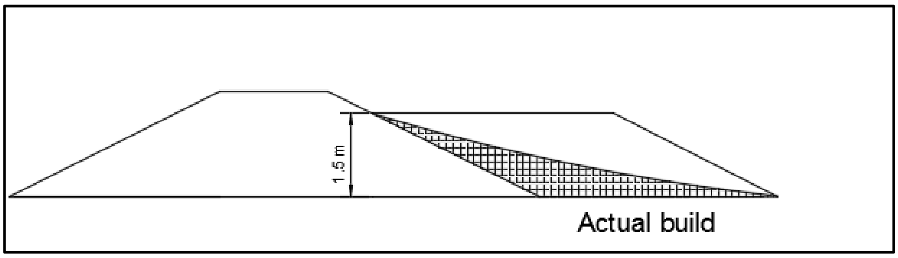


Figure 51: Cross-section of accretion.

It is assumed that the only coarse material in the longshore transport is the annual supply of 150 m^3 from the Swartkops river. In Chapter 6 it was stated that the yearly longshore transport is 200 000 m^3, which means that 0.075% (150 divided by 200 000) of the longshore transport consists of coarse material. Therefore, the focus will be on the sand accretion against the groyne.

The maximum height of the groyne is 4.1 m, which is equal to two times the armour and under layer diameters, with the determined armour and under layer unit diameter in the previous section. This means that up to the depth of 4.1 m below MSL, all the sediment will be obstructed by the groyne because the groyne will be higher than the water depth. Figure 52 displays the percentage of sand obstructed by the groyne at different depths.

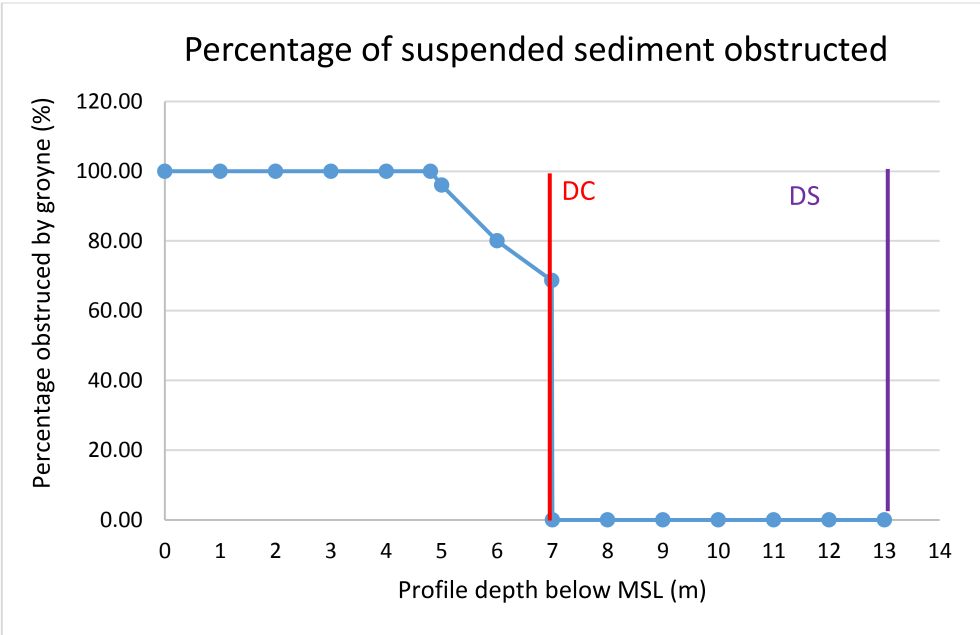


Figure 52: Percentage of sand obstructed by groyne.

In Figure 52 there is a sudden drop in percentage to 0%. This depth is at -7 m to MSL, which indicates the location of the head of the groyne as well as the depth of closure for cobbles (DC). The depth of closure for sand (DS) is also shown in Figure 52 above. Although the majority of the coarse material does not transport up to the depth of closure, this principle is also valid for sand. Therefore, the assumption is made that the percentage of material obstructed by the groyne is calculated up to the depth of closure for sand.

The total suspended sediment percentage obstructed by the groyne is 50% of the annual sediment transport. This means that the total annual volume of sediment obstructed by the groyne is 100 000 m^3. It was mentioned that the groyne was constructed without a core to increase the permeability of the groyne. This will result in an even lower value than 100 000m^3 but will have to be determined by means of physical modelling to get an accurate estimation.

The capacity of the groyne was calculated as 1 950 000 m^3, which means the capacity will be reached in 20 years (1 950 000/100 000). It is critical to understand that these values were based on an assumed accretion shape, which means this is not an exact representation of the capacity of the groyne.

7.2.6 General concerns

General concerns regarding the use of the submerged groyne are as follows:

- the obstructed sediment will consist of both sand and cobbles, further impacting the longshore transport;
- with a normal rubble mound breakwater, the core will facilitate dumping by truck, but for the submerged breakwater, this is not possible. This complicates the construction of the groyne; limiting it to water-based construction;
- the impact of the sand flow towards the sand-bypassing system is unknown;
- although this type of coastal structure is considered low in cost compared to other coastal structures, there is still a high-cost level involved (coastal structures are never inexpensive); and
- after a period, the groyne will reach its capacity, which will require clearance.

7.2.7 Concluding remarks

The submerged groyne poses the potential to be a viable long-term solution because of the low maintenance required, simple design and long life expectancy coupled with the design. However, one of the major problems with concept is that the groyne will obstruct not only the coarse material in the longshore transport, but also the sand. This will not be a problem shortly after the structure is installed, but will become a long-term problem when the sand accretes against the groyne. This process decreases the flow of sand toward the sand-bypassing system, which will lead to a shortage in sand supply.

The submerged groyne removes the coarse material from the sandtrap, which is the main objective of the conceptual solution, but to what extent the groyne will impact the surrounding shoreline is unknown.

7.3 Piles-and-mesh

7.3.1 Introduction

The pile-and-mesh structure is based on the same foundations as the groyne solution, but focuses on negative aspects to minimise them. The major concern of the groyne was the interference with the longshore sand transport. The obstruction is caused by the low permeability of the groyne, but if a higher permeability can be achieved, this concept can be very successful.

This concept can be achieved through the use of steel mesh screens with a punch diameter of 150 mm, which will allow smaller particles to pass through but will still restrict the larger sediment particles. The problem is that the screens cannot provide their own support, which is why the piles are used.

Piles are used for foundations where the material directly under a structure is not capable of supporting the load of the structure (Scott, 2016). In this case the piles will provide the necessary support to keep the steel screen upright, as illustrated in Figure 53 below. The high permeability of the mesh will also minimise the forces of the waves on the structure.

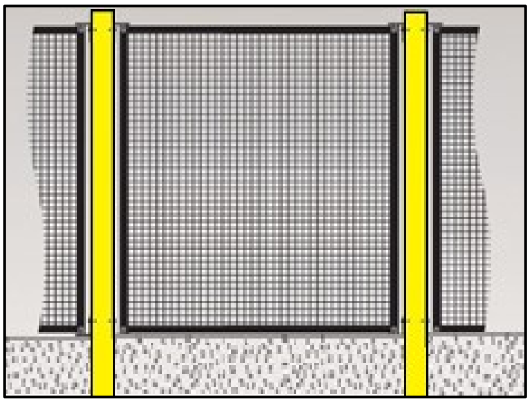


Figure 53: Pile-and-mesh concept.

The structure will only prevent the coarse material supplied by natural sources from migrating into the sandtrap. As stated in Chapter 4, the rock revetment and the remnants of the temporary construction works will still serve as a source of coarse material. The additional adjustments required to prevent these two sources will be explained Chapter 8 below.

7.3.2 Location

The submerged groyne obstructed a large volume of the longshore transport, which required the structure to be placed far enough from the sand-bypassing system to impact the flow of sand towards the system as little as possible. However, in this case the mesh is highly permeable allowing the majority of sand to move through the structure; resulting in a location much closer to the sandtrap.

Figure 54 below shows the layout of the sandtrap together with the 20 m-wide apron on the edge of the sandtrap. The pile-and-mesh structure will be located on the edge of the sandtrap apron as illustrated in Figure 54 below, which is located 46.5 m in the western direction from the centre line (CL) of the jet pumps.

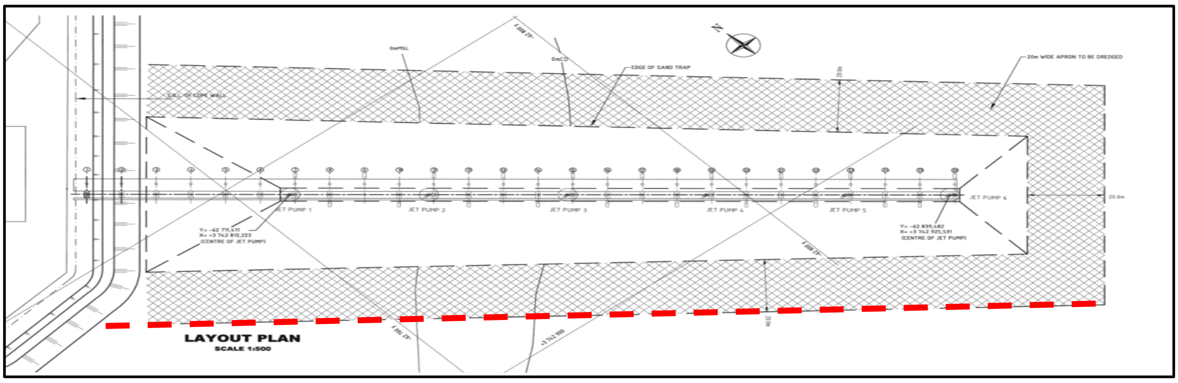


Figure 54: Location of the pile-and-mesh structure (Transnet, 2013).

7.3.3 Design

The pile-and-mesh structure will extend to a depth of 7 m (depth of closure for cobbles with a diameter of 150 mm), which is located about 230 m offshore. The landward end of the structure will extend up to the revetment, as shown in Figure 54 above.

Both the pile and mesh components will consist of steel, which will make them susceptible to corrosion in the coastal environment. Corrosion is mainly initiated by chloride ions present in the sea salts (Aluminium Federation of South Africa, 2011).

Several methods can stall or prevent this process, but the general and most cost-effective method is hot-dip galvanising the components. A protective zinc coating is applied by immersing the components in a bath of zinc. The zinc acts as a barrier between steel and the atmosphere; preventing corrosion. The design of each element of the structure is described in the following section.

7.3.3.1 Mesh

The steel mesh will consist of a rigid outer steel frame, which will support the inner woven steel components and add weight to the mesh as shown in Figure 55 below. The frame, with a thickness of about 100 mm, will also provide the necessary structure to fit into the H-beams as illustrated in Figure 56 below. The space between the woven steel will be 0.15 m in order to prevent coarse material with a diameter of 150 mm from passing through.

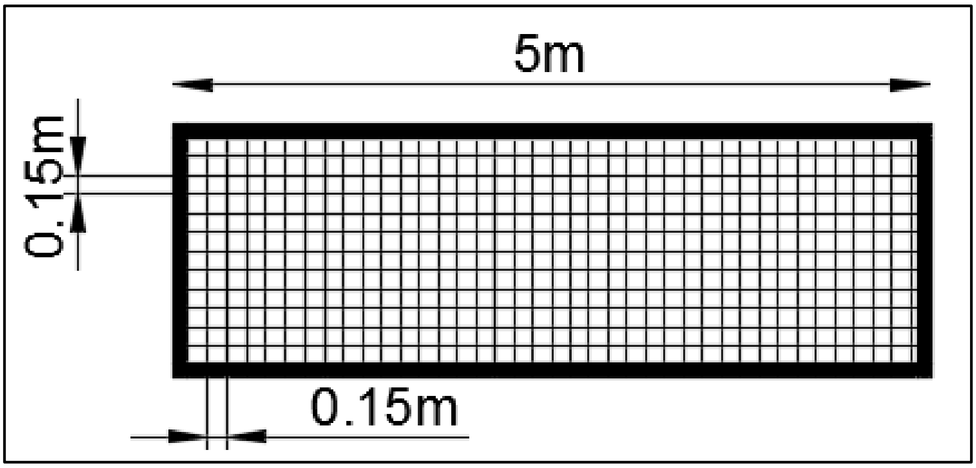


Figure 55: Mesh structure.

Height

In Section 5.3 it was stated that a bedload particle usually moves within a region of less than 10 to 20 times the particle diameter (Chanson, 2004), and it was assumed that a conservative lower limit of 10 should be used for this study. Taking this into account, the minimum height of the pile-and-mesh structure must be equal to 10 times the particle diameter of 150 mm, which is equal to a height of 1.5 m.

Scouring, the removal of granular material in the vicinity of a coastal structure caused by hydrodynamic forces (Hughers, n.d.), will occur at the base of the mesh, which will require additional height to reach the required height. The structure will also sink under its weight. In order to take all these factors into account the structure must be modelled to determine the required height.

The scouring that will occur is a positive aspect; providing more support to the structure as it buries itself in the sand. After a given period, the mesh will reach the maximum depth, which will be the final depth of the mesh.

Length

The length of the mesh will depend on both the accessibility of the component and the forces acting on the length of mesh. A longer mesh length will require larger/stronger piles, which will unnecessarily increase the cost of structure. The removal process of the shorter mesh will also be much easier. It is also necessary to make the screens long enough so that machinery (for example a bulldozer at the onshore section) can easily operate between the piles. A screen length of 5 m, as shown in Figure 55, will satisfy all of these factors.

7.3.3.2 Pile

Type

The type of pile used for this structure is a steel H-beam. The reason for choosing this specific pile was primarily for the shape of the pile. The H-shape will provide the support to keep the mesh screens in place as shown in Figure 56 below. At the beginning and the end of the structure, a channel pile will be used, as illustrated in Figure 56 below.

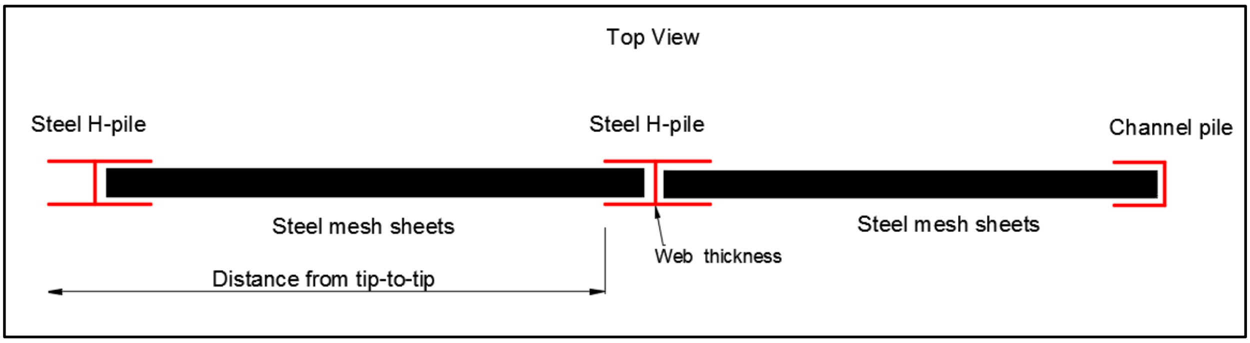


Figure 56: Top view of pile-and-mesh structure.

The use of steel piles also has other advantages, including that it is simple to install, easily acquirable, that its length can vary according to needs, and that it can provide immediate strength.

Placement

The distance between the piles will depend entirely on the size of mesh screens that will be used. For the case of the mesh sheet with a length of 5 m, the distance from tip-to-tip as shown in Figure 56 will be the sum of the length of the mesh (5 m), the web thickness of the H-beam, and approximately 5 cm for human error and manoeuvrability during installation.

Scouring

Scouring will also occur around the piles of the structure and is directly dependent on the diameter of the pile (Hughes, n.d.). Therefore, the scouring around the piles can only be determined once the dimensions of the piles are determined. The structure must be modelled in order to determine the required specifications of the piles.

Length

As mentioned in the introduction, the piles are not supporting a vertical load from a structure, which means it is not necessary to extend to bedrock. Because the mesh is highly permeable, the vertical force will be relatively low. The total length of the pile need to take the following factors into account: the mesh height (as previously stated), the scouring around the pile and the necessary support to withstand the wave action against the structure. The structure must be modelled in order to determine the resulting length of the pile.

Capping

To ensure the steel frames stay in place, a hole can be drilled through the top of all four the flanges as shown in Figure 57 below. It's critical that the galvanisation process is applied after the holes are drilled. Steel bolts (also galvanised) can be screwed from one end of the flange to the opposite flange to prevent the steel frame from lifting out of the H-beam until it sinks to the maximum depth.

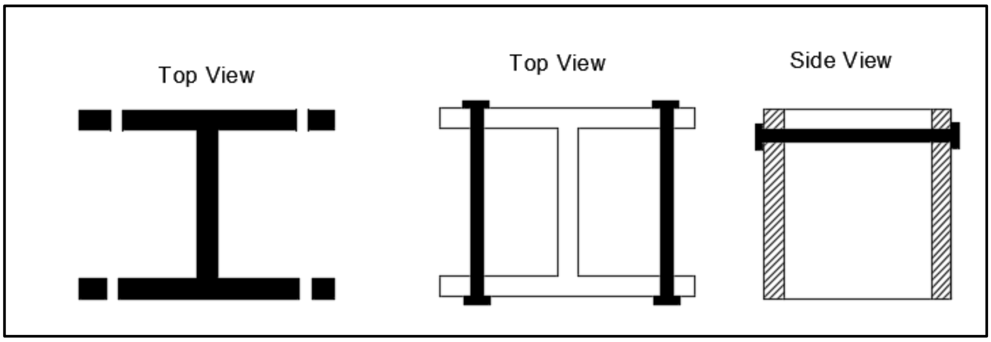


Figure 57: Capping of the H-beam.

7.3.4 Installation

The accretion against the western breakwater allows for land-based installation of the piles during LAT for the majority of the structure. As stated in section 6.4.4, the layer that the pile will be driven into will consist of sand. This means that the steel piles can be driven into the sand with the help of a small drop hammer or vibrating hammer (care must be taken not to damage the galvanised layer). The installation in the deeper sections of the structure will have to be installed by sea-based method, which can be done from a vessel.

The accuracy of the location of each pile will be a crucial aspect of the installation for the frames to fit correctly. Such locations can be ensured by GPS coordinates and a pile-driving template. After the installation of the piles, the steel frames can be slid into the H-beams. The occasional manual excavation will be necessary at the onshore locations to place the capping on the H-beam.

7.3.5 Capacity and maintenance

The main concept behind this potential solution was to create a structure with low maintenance and a small impact on the longshore sand transport. These features created a high capacity for the structure, which is why it was not designed to be removed on a regular basis.

The precise capacity of the structure is challenging to determine, but through visual observation of the height of the accretion against the screen, it can be determined when the structure is nearly reaching the maximum capacity.

In Section 6.4 it was stated that the majority of the coarse material was found at +1.5 m and -1.5 m to MSL. This means that the section of the structure where capacity will be reached first, will be at this section. MSL is at -1.026 m to CD (also LAT), which means that the majority of this section can be cleared by land-based method during LAT.

The first step of the land-based method is to remove the screens from the piles with a crane. The distance between the piles was designed to allow a bulldozer to remove the coarse material that accumulated against each screen.

The removed material, which will consist of both sand and coarse material, will be unloaded into a suspended steel basket with a punch diameter of 100 mm (adopting a conservative approach). This will allow all particles with a diameter smaller than 100 mm to pass through and will retain only the coarse material. This process will take place on the beach, which will allow the smaller particles to be placed back into the coastal system.

If it becomes necessary to remove some of the screens in the deeper section of the structure, it will have to be done by sea-based method. A floating crane, which is a crane on a vessel, can be used to remove the screens. After the removal of the screens, a grab dredger can be used to clear the obstructed coarse material. The same suspended steel basket can be used to retain the coarse material.

There will also be marine growth (accumulation of micro-organisms, plants, algae, or animals on a wetted surface) on the mesh screen, which will decrease the permeability of the screens over time. Such material will have to be removed on-site during LAT, or the screens can be removed and cleaned. Through visual observation of the screens, it can be determined when this process is necessary.

7.3.6 Expansion

The structure was designed specifically to obstruct the coarse material in the longshore transport, but it is possible to install the structure around the entire sandtrap of the sand-bypassing system. The structure will be constructed on the apron of the sandtrap as shown in Figure 58. This will prevent the coarse material originating from the revetment from migrating into the sandtrap.

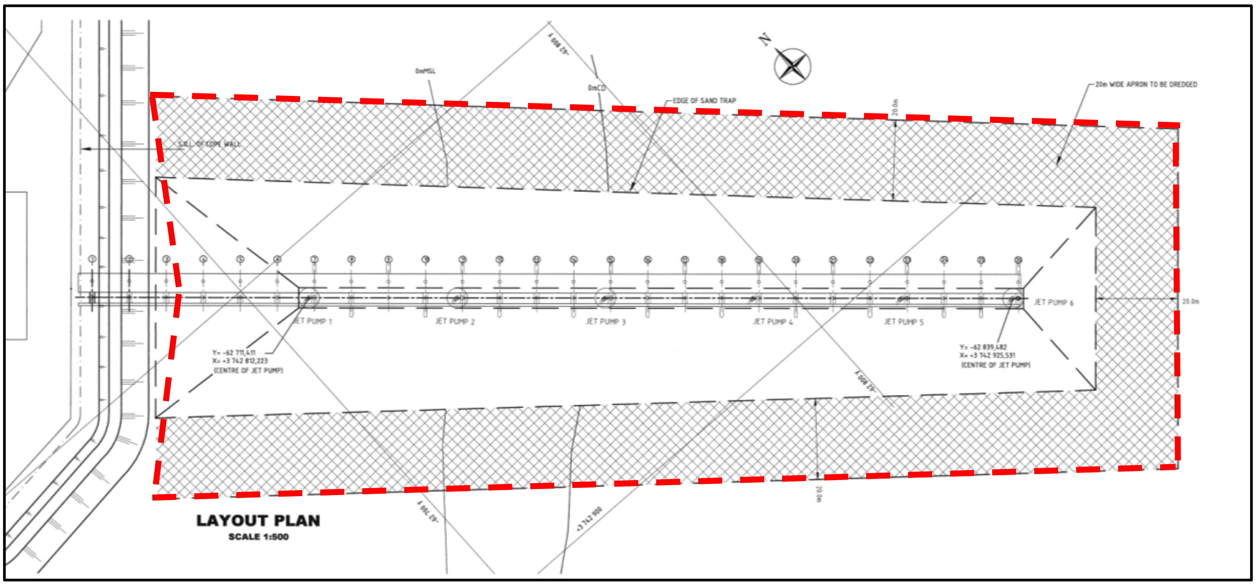


Figure 58: Pile-and-mesh around sandtrap (Transnet, 2013).

The movement of the larger armour units may cause breakage in the screens, which means that the state of the structure must be inspected regularly.

7.3.6 General concerns

Potential challenges related to the use of this structure are as follows:

- the performance of the structure is unknown because this is not a general coastal structure (design is based on theory);
- the removal of the screens might be challenging due to the weight of the accumulated coarse material;
- the removal of the screens with a crane might damage the frame of the mesh structure;
- corrosion might still be a problem if the galvanisation process was not performed properly (life-expectancy is not precise) but each component can easily be replaced if corrosion causes the structure not to perform according to the design;
- the installation method by means of drilling or vibration might damage the outer layer of the galvanised components, which will cause corrosion;
- if the beams are not installed in the precise location, the placing of the screens will become challenging;
- during removal of the screen, the weight of the coarse material might cause breakage in the mesh screens;
- marine fouling will decrease the permeability of the structure; and
- while the structure will be clearly visible in the water, it is unknown how marine animals will respond to the structure.

7.3.7 Conclusions

The pile-and-mesh structure can be considered a viable solution for the migrating coarse material. The concerns mentioned in the section above can easily be resolved using numerical or physical modelling of the system. The other concerns can be prevented by following thorough inspection before, during and after the installation of the structure.

The advantage of using the smaller screen is that if a single screen is defective, only that particular screen needs to be replaced, which means that the whole system will not be interrupted. However, the system will not function effectively if all the mesh screens are not in perfect condition. It is therefore of critical importance to always have additional replacement components readily available.

Compared to the submerged groyne, the impact of the structure on the longshore transport is effectively reduced by only obstructing the material causing problems at the sand-bypassing system. The design of the system was also done in a manner to ensure the installation methods involved remain as simple as possible. Overall, this system can perform as a very efficient solution to the problem at hand.

7.4 Mobile jet pump

7.4.1 Introduction

The previous conceptual solutions all aimed to prevent coarse material from migrating into the sandtrap, which required not only additional construction but also certain modifications to the rock revetment around the fixed bypassing system and the clearance of remnants of the temporary works. This solution will not interfere with the migration, but will rather deal with the coarse material after it is already located in the sandtrap.

In Section 3.2.7 it was mentioned that coarse material builds up in the bottom of the jet pump cones. This process prevents the sand from being fluidised, which decreases the production rate of the sand-bypassing system. This solution will use this process as a positive aspect by approaching it as the accumulation of all the coarse material from multiple sources at one location.

A mobile jet pump with a large enough intake will be used in combination with the jet pumps' intake cages to remove all the coarse material accumulated at the bottom of the cones. The intake cages will prevent further blockages in the permanent bypassing system, which will reduce down-time caused by cavitation issues and will, in effect, increase production.

The clearance of a cone will commence when a jet pump reaches the point when no further bypassing is possible. This allows for the maximum amount of sand to be removed before the dredging is required; reducing the volume of sand dredged by the mobile system.

The advantage of such a system is that only one large jet pump is needed to clear the debris at all six jet pumps. The advantage of this solution is that except for replacing the armour layers where the core is exposed, no other alteration is required for the solution to function.

7.4.2 Access

To reach the coarse material located in the sandtrap, the jet pump system requires access. This can be achieved by either a land or sea-based method. In the previous dredging works that were completed at the Port of Ngqura in 2015, the jetty of the sand-bypassing system was used to gain access (Jansen, 2016). Using the jetty again will therefore likely provide the easiest and most reliable access for the dredging machinery. As there are no vertical obstructions on the sand-bypassing jetty, the machinery will be able to rotate freely in the horizontal plane on the jetty.

7.4.3 Design

In this section the components of the mobile jet pump will be discussed. Due to the costs involved in the procurement of some of the components, the exact cost of each component was unobtainable. Therefore, it was opted to obtain second hand costs in order to get an indication of the costs involved.

7.4.3.1 Jet pump

The reason for specifically choosing a jet pump for the dredging equipment is two-fold, namely that it will be the safest method to dredge without damaging the existing bypassing system; and certain elements of the mobile system is already provided for by the existing system.

In the dredging works mentioned in Chapter 3, the coarse material that was removed from the sandtrap varied in size up to 450 mm. Most of the coarse material removed by the airlift pump reached a size of 300 mm, and the larger sediment was removed by divers. Therefore, the intake opening of the jet pump will be 300 mm, which will remove the majority of the coarse material accumulated in the cones. The larger particles, if any, can be removed by divers. An example of a pump with these characteristics is a Damen DOP® pump, which is a submersible dredge pump (Damen, 2016).

A pump with the required characteristics as stated above have an estimated second hand price of R 2 million according to Dredgebrokers (2016), which the condition is unknown. This price only includes the submersible pump and the motor. The cost of the other required components will be given in each section.

7.4.3.2 Crane

The width of the jetty is about 8.5 m (derived from drawings that are not to scale). Unfortunately, the entire width of the jetty is not available for access due to the service section on the jetty. The width available on the jetty is about 4 m (derived from drawings). This passage will be the maximum width of the crane that can be used.

The single jet pump will be mounted on a small crane/boom with a maximum reach of 7 m (4.5 m to extend past the width of the jetty, plus the additional 2.5 m for the furthest point of the base of the sandtrap). The reason why a smaller crane is required is because it is only necessary to clear the coarse material located under that specific jet pump as illustrated in Figure 59 below.

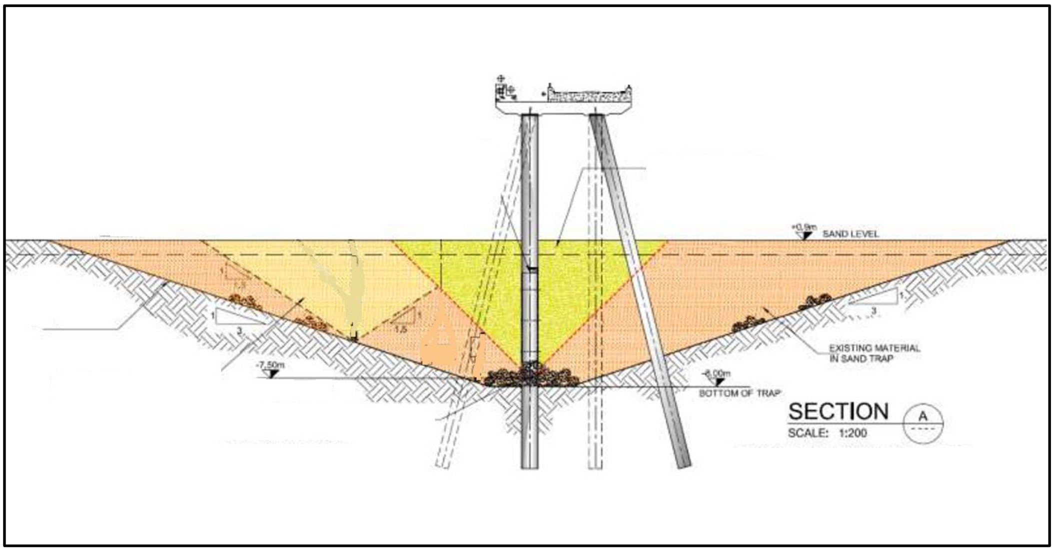


Figure 59: Coarse material accumulation (Transnet, 2013).

The jet pump will be lowered by suspension from the crane to the correct position, as shown in Figure 60 below. The extension at the tip of the crane, as shown in Figure 60, will allow the suspended dredge pump to access the section directly below the jetty. It is not a requirement that mobile jet pump must be able to reach every location in the sandtrap because of routine dredging that will occur, which will only remove the coarse material accumulated beneath the permanent jet pumps and not the entire sandtrap. A second hand boom truck with a reach of 20 m is estimated at R 465 000 according to OnlineCranes (2016), which the exact state of the crane is unknown.

Figure 60: Mobile jet pump crane (Damen, 2016).

7.4.3.3 Power

In dredging works of 2015, the Port of Ngqura supplied the power for the dredge pump. Therefore, this source will be used during coarse material dredging (Jansen, 2016). The alternative method is to procure a portable generator, which can also power the mobile jet pump.

7.4.3.5 Pipes and hoses

The clear water supply of the permanent jet pump will be used for the mobile jet pump. The pipelines supplying the water to the location of the mobile jet pump must be long enough to reach every location on the sand-bypassing jetty.

The pipeline connecting the jet pump to the discharge location must be slightly larger than 300 mm in order to reduce unnecessary cavitation issues. The pipeline must also be flexible in form and wear-resistant in order to adapt to the movement of the suspended jet pump.

The length is made up of the following sections:

- the height of the jetty, which is 18 m above the deepest point in the sandtrap;
- the reach of the crane, which is 7 m; and
- the additional 18 m to extend to the discharge point above the neighbouring jet pump (the maximum distance between the jet pumps is 36 m).

This results in a total pipe length of about 45 m. A 400 mm dredge pipe for discharge or suction, with flexible properties is estimated at R 7 300 per meter. The total of 45 m that is required will cost R330 000 (Dredgebrokers, 2016).

7.4.3.6 Dredged material

The dredged material will be pumped back in the direction of the jetty and then along the jetty towards the neighbouring jet pump. It will be discharged into a metal basket suspended over the neighbouring jet pump in order to remove the coarse material from the slurry mixture. The basket, with 100 mm openings (using a conservative approach), will allow the smaller particles to pass through while still obstructing the larger particles. The residual material will be unloaded onto a truck and removed from the jetty.

7.4.4 General concerns

Challenges pertaining to this proposed solution include:

- the cost involved in the procurement of all the equipment could be prohibitively high;
- mobile jet pumps experience numerous cavitation issues, which will result in frequent downtime and cost;

- certain adjustments to the permanent system are required (extension of clear water and power supply) in order to accommodate the mobile system; and
- the mobile system will require trained people to handle this difficult operation.

7.4.5 Conclusion

The procurement of second hand components resulted in a total cost of over R3 million. Assuming that these components will be new when bought, will result in a much higher cost. Therefore, mobile jet pump solution is an unlikely solution due to the high costs involved in the procurement of the jet pump and all the additional elements. The overall concept is a good solution to the problem because there will be no further construction required and the impact on the environment will be minimal.

The advantage of this solution is that the regular dredging works to clear the sandtrap will no longer be an expense for the Port. Another option to make this solution financially viable is to procure the equipment with another company/port because this equipment will not be used continuously.

While there are numerous positive aspects coupled with this proposed solution, the financial implications related to this proposal are so significant that it does not appear to be feasible option.

7.5 Coarse material catchnet

7.5.1 Introduction

The coarse material catchnet (henceforth referred to as 'catchnet') concept is somewhat similar to the pile-and-mesh structure by focusing on the negative aspects of the submerged groyne and by increasing the permeability of the structure. The difference between the catchnet and the pile-and-mesh structure is that the catchnet itself will be able to remove the coarse material from the coastal system and will be designed for routine clearance.

The design of the catchnet is based on the design of a gillnet, which is used for bottom fishing. A gillnet is one large mesh, or a series of mesh panels, that creates a vertical wall along the seabed (NOAA fisheries, 2014). The groundline (bottom line) is weighted down by lead weights to remain in permanent contact with the seafloor. The groundline can also be made of a weighted "foot rope", which is lead cored rope that serves the same purpose.

The floatline (upper line) is held up by small floats, usually cylindrical in form and made of solid plastic, and which is evenly distributed along the floatline to keep the net upright (NOAA fisheries, 2014). The mesh size of the net is determined by target size fish. Anchors attached to the end of the anchor rope keep the net stationary. An example of a gillnet layout with its respective components is illustrated in Figure 61 below.

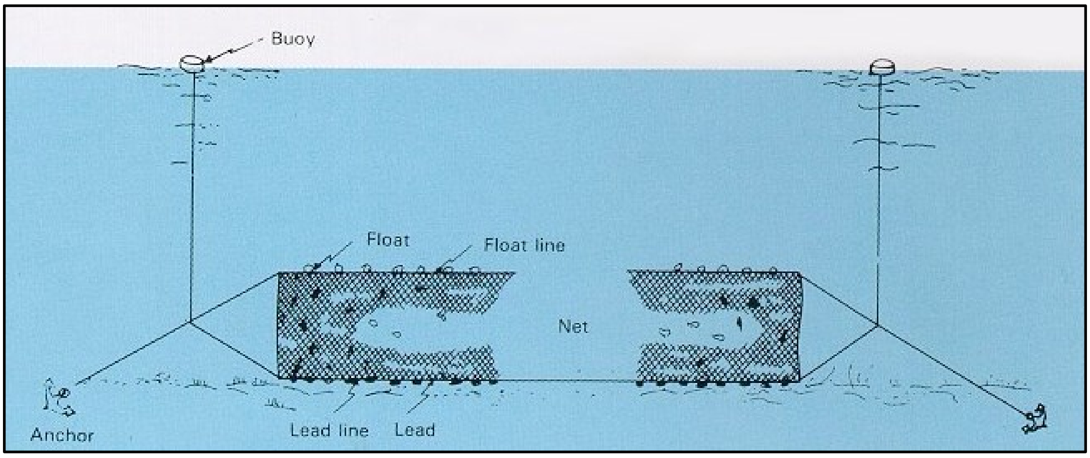


Figure 61: Gillnet layout with components (DHgate, 2016).

Individual elements of the catchnet will differ from the design in Figure 61 above. The first major adjustment is to add a net extension to the groundline to trap the accumulated coarse material. The general design of the catchnet is shown in Figures 62 to 64 below. This added feature will also allow the catchnet to retain the obstructed coarse material during removal.

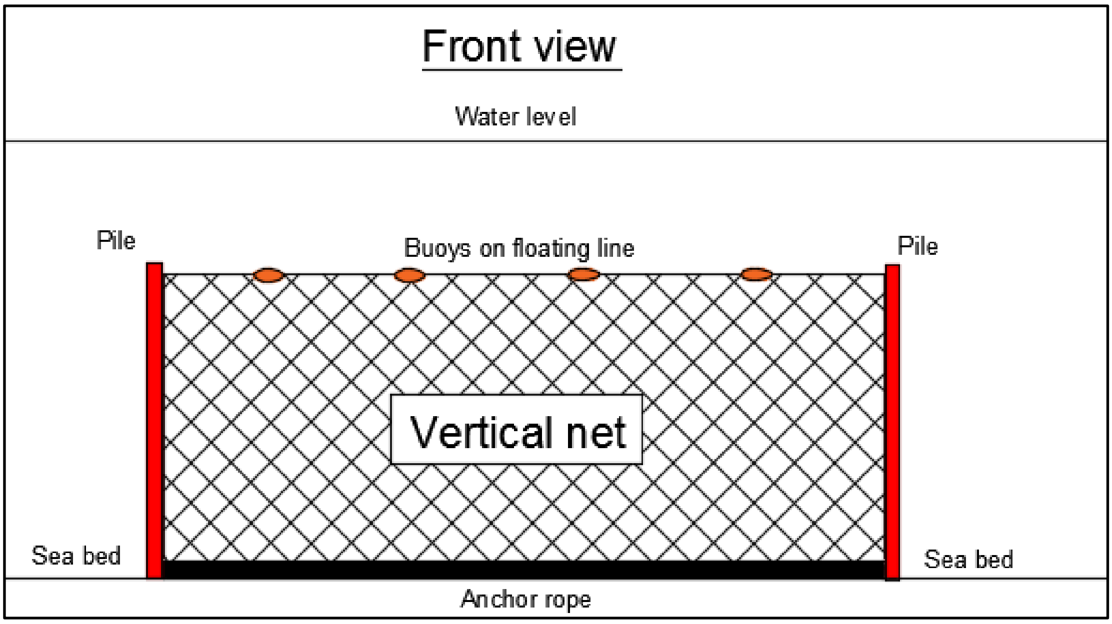


Figure 62: Front view of catchnet.

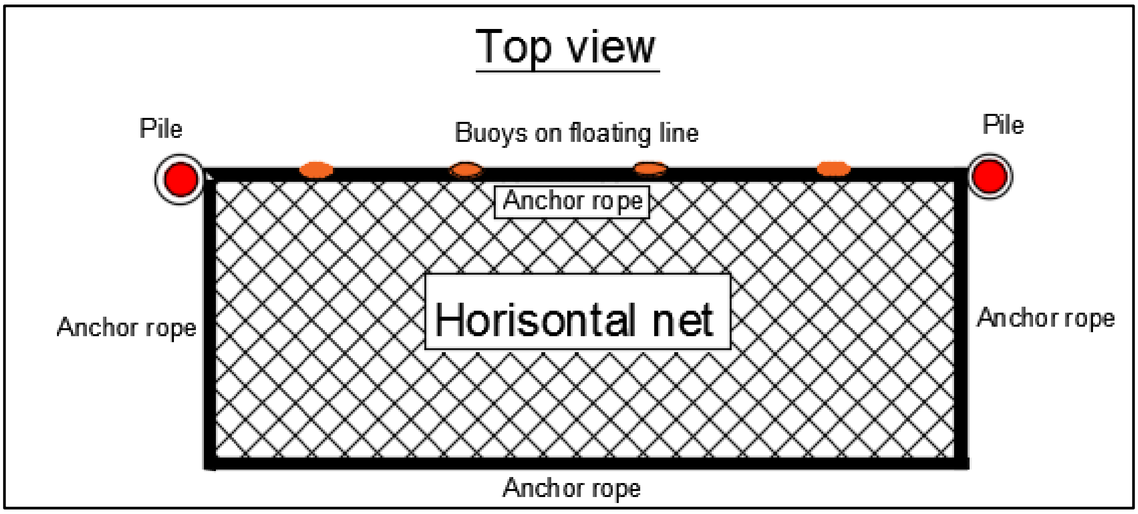


Figure 63: Top view of catchnet.

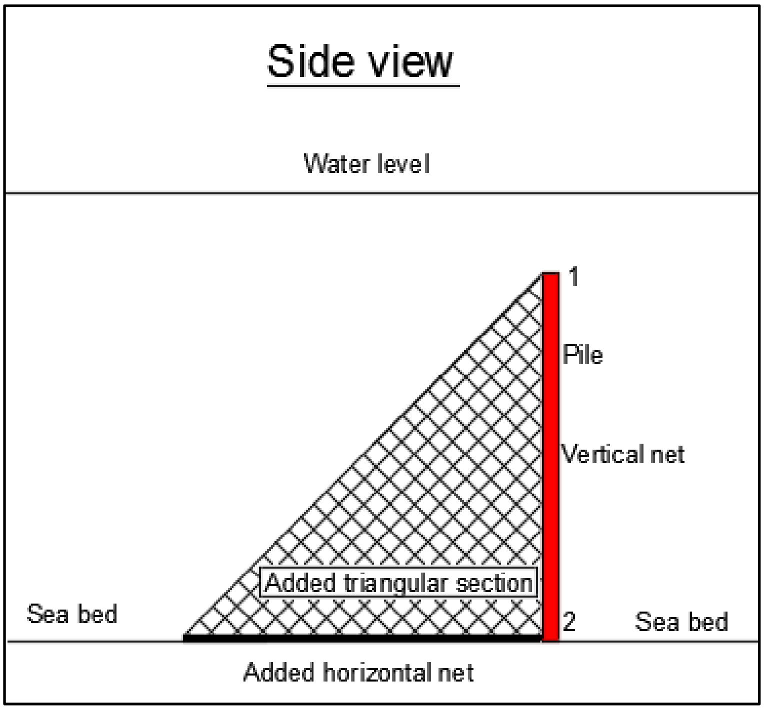


Figure 64: Side view of catchnet.

The gillnet is usually upheld by floating devices on the floating line. This is the other characteristic that will be modified because the weight of the coarse material that accumulates against the vertical net will cause the net to sag. An alternative approach is therefore used with the help of piles, which will ensure that the net stays upright (Figures 62-64).

The net component of a gillnet is made of transparent monofilament line to reduce the visibility of the net. The more visible the net, the less of an impact it will have on the marine environment. As visibility is not an objective of the catchnet, and the protection of the marine environment is paramount to the project, the complete opposite approach will be used in the design.

7.5.2 Location

The permeability of the catchnet and the pile-and-mesh structure is very similar, which means that the exact location of the pile-and-mesh structure can be used for the catchnet. This location is on the apron of the sandtrap, as shown in Figure 65 below.

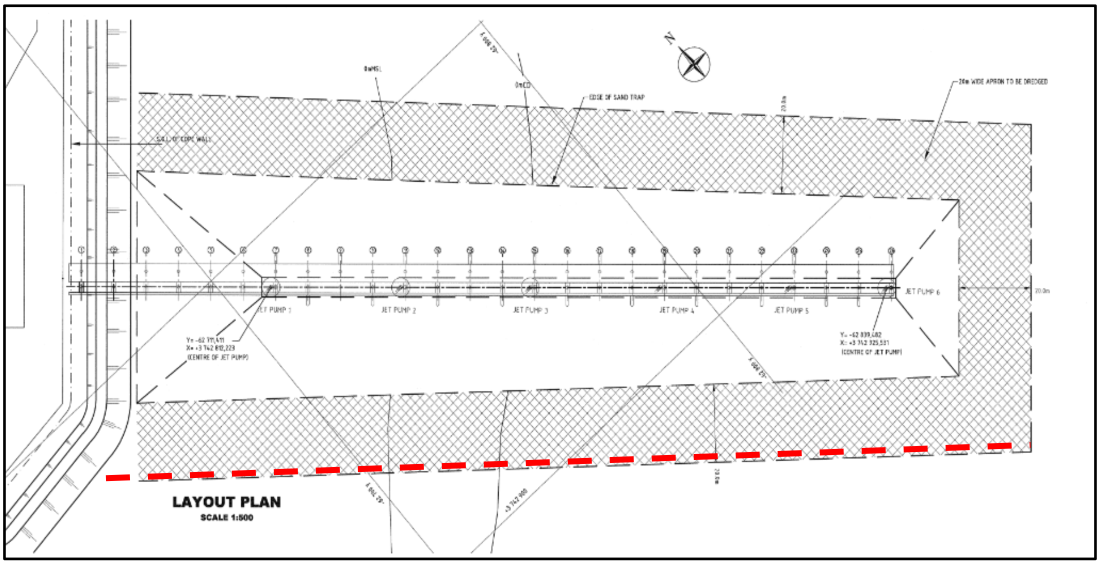


Figure 65: Location of catchnet (Transnet, 2013).

7.5.3 Design

The catchnet will extend to a depth of 7 m (depth of closure for cobbles with a diameter of 150 mm), which is located about 230 m offshore. The landward end of the structure will extend to the revetment as shown in Figure 65 above.

The pile components will consist of steel, which will make them susceptible to corrosion in the coastal environment. Every pile must therefore be galvanised before installation. The net component consists of ultra-high molecular weight polyethylene fibre (for example, Dyneema® fibre). According to Marlow Ropes (2012), the Dyneema® fibre has the following characteristics:

- high in strength (15 times stronger than steel wire);
- light in weight (15 times lighter than steel wire and it floats in water);
- water resistant;
- chemical resistant (performs well in dry, wet, salty and humid conditions); and
- ultraviolet (UV) resistant.

The frequent use of nylon nets for salmon farming is being replaced by Dyneema® nets because of these properties. Similarly, the Dyneema® fibre will ensure that the net component will also be able to withstand the site conditions at the sandtrap. The high strength lines also have extremely low elongation and high cross-sectional stability, which will be very beneficial during the removal of the net.

7.5.3.1 Net design

The net will be an ultra-cross knotless net, which is a net connected by the interweaving of ropes as depicted in Figure 66 below. By avoiding knots throughout, the net will decrease the overall weight of the net.

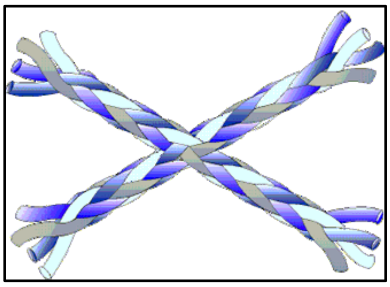


Figure 66: Knotless netting configuration (NETsystems, 2014).

According to NETsystems (2014), the largest thread diameter that can be used for a 100 mm mesh is 7 mm. The reason for using a 100 mm mesh in the design is that during the removal of the net, the weight of the coarse material will force some of the openings to enlarge. To prevent the 150 mm particles from remaining trapped, a 100 mm mesh will be used, which is a conservative approach. The single rope of a Dyneema® fibre with a diameter of 7 mm has a weight of 27 g per meter and a breaking strength of 4600 kg (Dyneema, 2015).

The total weight of the annual coarse material transported along the coast is 318 000 kg on land for quartzite, with a density of 2650 kg/m^3 (CIRIA, 2007). The submerged weight of the same volume is 198 000 kg. It is assumed that the coarse material is evenly distributed along the catchnet and the catchnet will be removed every month. The weight per meter net is equal to 198 000 divided 12 (months) divided by the total length of 230 m, which delivers a result of 72 kg per meter of the net.

The accretion of coarse material against the net will be similar to the accretion against the groyne, but it is unknown what fraction of the weight will lean against the net, causing it to sag. Due to this lack of information, it is roughly assumed that a quarter of the total weight will lean against the vertical net. This results in a weight of 18 kg per meter of the net (72 divided by 4).

The vertical section for the 10 x 1.5 m net will have a weight of about 8 kg without taking the threading on the edges into account. This is determined by using a spacing of 100 mm between each rope in the vertical and horizontal direction, resulting in a total length of rope for the 10 m net of 300 m. The weight of 7 mm rope is 27 g per meter, which results in a total weight of 8 kg for the 10 m net. In order to take the threading on the edges into account a conservative approach is followed with the overall weight of the net taken as 9 kg. The required weight to be upheld by the buoys is the weight of the vertical net plus the weight of the coarse material leaning against it; resulting in a total weight of 188 kg for the 10 m net. The 50 x 1.5 m net will have a total weight of 942 kg.

The floating line of both nets will be upheld by HTM series floats with a 1 m spacing between each float. Using this configuration, each buoy must be able to uphold a weight of 19 kg, which means that a HTM-2 buoy can be used with a buoyancy of 17 kg (Polyform, 2016). Although the buoyancy is smaller than the required weight, there will also be an upward force exerted by the piles on the edges of the net, which will be sufficient. This means that nine buoys will be used on the 10 m net and 49 on the 50 m net.

Anchor ropes will be used along the edges of the horizontal net to ensure the horizontal section remains in a fixed position. The anchor rope at the connecting corner of the horizontal and vertical net must counterbalance the upward force of HTM buoys minus the weight of the net. The lead-line that will be used in this section must therefore have a submerged weight 14 kg per meter. The weight of the lead-line along the other edges of the horizontal net can be much lower because the upward force of the buoys will not affect these sections. It is assumed that a lead line of 1 kg per meter will be used along these edges because this will be sufficient to bury the corners into the sand.

An added loop with a diameter of 200 mm will be weaved into the net at the bottom and top corners of the vertical net at locations 1 and 2 in Figure 64 above. The use of these loops will be explained in the capping section. Pingers, which are small acoustic devices emitting high pitched sounds, will also be added to the floatline in order to reduce bycatch (marine species that are caught unintentionally).

Height

In Section 5.3 it was stated that a bedload particle usually moves within a region of less than 10 to 20 times the particle diameter (Chanson, 2004), and it was assumed that a lower limit of 10 should be used. Taking this into account reveals that the minimum height of the catchnet must be equal to 10 times the particle diameter of 150 mm, which is equal to a height of 1.5 m. Although the net will experience sagging at certain sections, this will be minimal because of the upward force of the piles and buoys.

Length

The entire catchnet will consist of a combination of short and long nets. The shorter nets will be 10 m in length and will be used at sections where the sediment transport rates are high. The reason for using shorter nets is because the total weight carried by the net will be larger at these locations, which will result in a larger weight during removal. The long nets will be 50 m in length and will be used in the section where the sediment transport rates are lower.

In Chapter 6 it was mentioned that the majority of the sediment (sand and coarse material) transports above – 4 m to CD or -5m to MSL Therefore, the shorter nets will be used up to this depth, which is approximately 130 m offshore. Two 50 m nets will be used to cover the remaining 100 m.

7.5.3.2 Pile

Type

The type of pile that will be used for this structure is a steel pipe pile. The reason why this specific pile was chosen was primarily for its shape. The circular shape will ensure fast and easy installation/removal of the net. The loops on the corners of the net will slide over the steel pile in order to keep the net in place. Figure 67 and Figure 68 below show the concept of the connection between the net and piles.

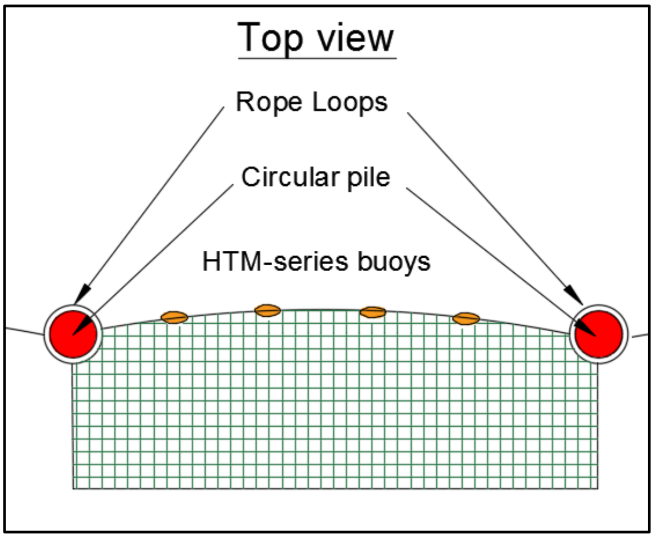


Figure 67: Top view of the circular pile and added loops.

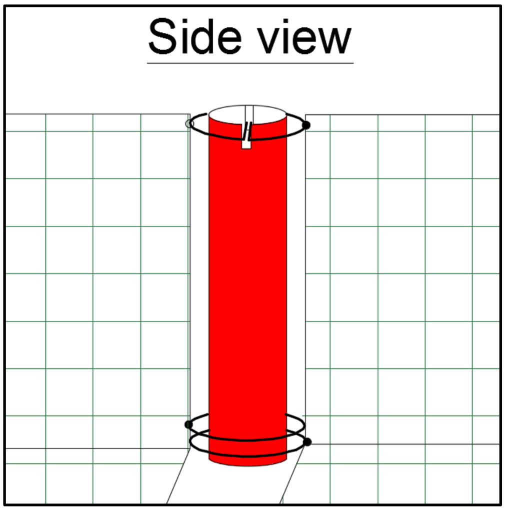


Figure 68: Side view of pile and net connection.

Placement

The distance between the piles will be determined by the net length connected between the two piles. As previously mentioned, the two lengths that will be used are 10 and 50 m respectively. The piles will be spaced 10 m apart up to a depth of -4 m below CD, which is about 130 m offshore. From there, two more piles will be installed 50 m apart from one another.

Scouring

Scouring will also occur around the piles of the structure and is directly dependent on the diameter of the pile (Hughes, n.d.). Therefore, the scouring around the piles can only be determined once the dimensions of the piles are determined. The structure must be modelled in order to determine the required specifications of the piles.

Length

As mentioned in the introduction, the piles are not supporting a vertical load from a structure, which means it is not necessary to extend to bedrock. Because the net is highly permeable, the vertical force will be relatively low. The total length of the pile need to take the following factors into account: the net height of 1.5 m, the scouring around the pile and the necessary support to withstand the wave action against the catchnet. The structure must be modelled in order to determine the resulting length of the pile.

Capping

To ensure that the top loops of both nets remain in place, the circular pile will be slightly modified. A groove of about 50 mm deep will be made into the top of circular pile. The upper end of the pile will have screw indents in order to place a cap on top of the pile. This will ensure that the added loops will not move out of the pile. The properties of the upper end of the pile are shown in Figure 69 below.

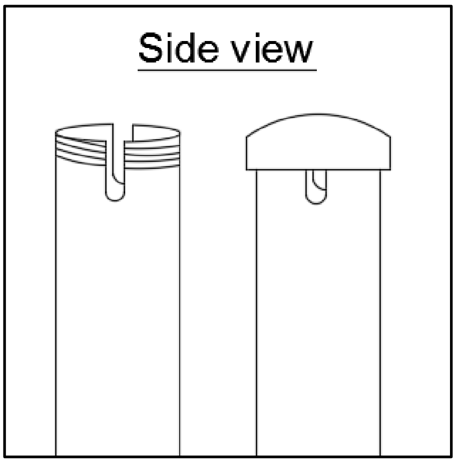


Figure 69: Circular pile capping.

The grooves in the pile will cause the rope to chafe, which will cause the rope deteriorate quicker. Therefore, the circular sections must be protected by an anti-chafe/rope protector which stops abrasion, wearing and chafing of the rope in order to prevent strength loss and maximize the lifespan. The galvanisations process must take place after these indents and grooves are made.

7.5.4 Installation

Accretion against the western breakwater allows for land-based installation of the piles during LAT for the majority of the structure. As stated in Section 6.4.4 above, the layer that the pile will be driven into will consist of sand. This means that the steel piles can be driven into the sand with the help of a small drop hammer or vibrating hammer. The installation in the deeper sections of the structure will have to be installed by sea-based method, which will be done from a vessel.

The accuracy of the location of each pile, which can be measured by GPS coordinates and a pile-driving template, will be an important aspect of the installation. After the installation of the piles, the catchnet must be set. Because one pile will provide support for two nets, the easiest method will be to install neighbouring nets simultaneously. The capping must be placed after both nets are set.

7.5.5 Hauling

The catchnets will be cleared one at a time in order to decrease the possibility of coarse material transporting past the catchnet. The net will be removed during HAT in order to allow the majority of sand to pass through the net, which will decrease the weight of the net. This means that the hauling of the net will be water-based. The type of vessel that will be used is a fishing vessel (trawler) because of the similar removal process required.

To remove the net, the four corners are taken together and the net is lifted out of the water, assuming that the nets are strong enough. The capacity is addressed in the following section). The side net (triangular section) will ensure that the coarse material stays trapped within the net. The entrapped material and the net will be loaded onto the vessel where it will be taken to shore. The replacement net will be set directly after the current net is removed. The removed net must be cleared, inspected and cleaned before it can replace the subsequent net.

The nets located near the revetment can be cleared by land-based method by using a bulldozer to lift the four corners simultaneously. During the lifting process, the majority of the smaller particles will fall through the openings, leaving the coarse material behind. This will be done on the beach, which will allow the smaller particles to be placed back into the coastal system.

7.5.6 Capacity and maintenance

The capacity of the net is difficult to determine because of the varying coarse material transport rates along the net. To avoid that the overall weight of the net becomes too high, however, a conservative approach is followed by removing the net every month.

The total weight of the net is the weight of the coarse material, horizontal and vertical net and both types of leadline. The total submerged weight of the 10 m net is equal to 364 kg, and the 50 m net is equal to 1758 kg. The weight of both nets out of the water (during hauling) is 494 kg and 2403 kg. As previously mentioned a single rope is able to withstand 4600 kg. Following a conservative approach by assuming the that the entire load of the net, 494 kg and 2403 kg, acts as a point load on a single rope, indicates that the net is able to withstand the load. The reason why this is a conservative approach is because it is unlikely that the entire weight of the net will act as a point load, but will rather be distributed.

The values above also indicate that the 50 m net will become the limiting factor when the removal frequency decreases. This is a highly unlikely occurrence due to the fact that the sediment transport rate at these sections is very low compared to the nets located above – 4m to CD. The removal cannot be based on the 50 m nets because the initial transport rates that were used for the design was for an evenly distributed transport rate, which is not the case.

In order to investigate the performance of the catchnet for more reasonable conditions, the assumption is made that the total annual volume of 150 m^3 is only obstructed by the nets above -4 m to CD and is evenly distributed along this section. This means that the 150 m^3 is only obstructed by the 10 m nets and not both the 10 and 50 m, which means smaller nets will carry a higher load. This results in monthly submerged weight for the coarse material load of 1269 kg on the 10 m net, with a total weight out of the water equal to 2245 kg. If the net is removed after two months, the total weight out of the water is 4283 kg, which is still below the breaking strength of the net. This shows that the net should be removed every two months, but this will cause the vertical net to sag as well. Therefore, it is recommended that the catchnet can be removed on a month to two-month basis, but never longer than two months. In addition to the monthly clearance, the net must be removed after every storm due to the high sediment transport rates experienced during these events.

7.5.7 Expansion

While the catchnet was designed to specifically obstruct the coarse material in the longshore transport, it is possible to install the catchnet around the entire sand-bypassing. The catchnet will be installed on the apron of the sandtrap as shown in Figure 70 below. This will prevent the coarse material originating from the revetment from migrating into the sandtrap. The catchnet will not be able to obstruct the largest armour units, because it will cause the catchnet to break, but will obstruct the majority of the displaced units. Therefore, a routine inspection of the catchnet is required to remove these large units from the catchnet as quickly as possible.

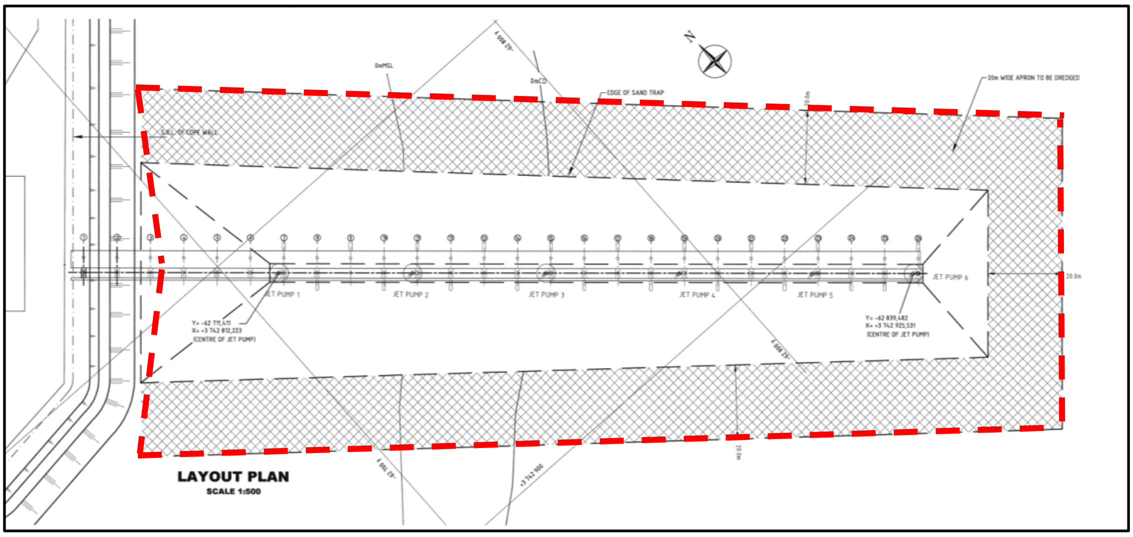


Figure 70: Catchnet around entire system (Transnet, 2013).

7.5.8 General concerns

Challenges related to this proposed solution include:

- certain sections of the net will spend the majority of the time above water (during LAT), which serves little purpose and will cause them to deteriorate quicker;
- even with the prevention methods applied, there will still be unwanted fish and other marine creatures trapped by the catchnet;
- the net component is not used in the general manner for which it was designed, which means that the performance of the net is unknown;
- the installation method by means of drilling or vibration might damage the outer layer of the galvanised components, which will cause corrosion;
- individual sections will have to remove larger volumes of coarse material, which means that they will deteriorate more quickly; and
- if the nets are not removed at a regular interval, the load on the nets will only increase, which will make the removal process challenging.

7.5.9 Conclusion

The catchnet can be considered a viable solution for the migrating coarse material because, at least theoretically, the coarse material will be removed. Although the performance of the catchnet is unknown, it can be determined by physically modelling the system in the same conditions. The other concerns, mentioned in the section above, can easily be prevented by following routine clearance methods and regularly inspecting the nets.

The impact of the catchnet on the longshore transport was effectively reduced compared to the submerged groyne by only obstructing the material causing problems at the sand-bypassing system. The system was also designed in such a manner that the installation and removal methods involved are relatively straightforward. The main benefit to using this system compared to the pile-and-mesh structure is that the coarse material can easily be removed from the coastal system. In theory, overall the coarse material catchnet can be a very successful solution, but the monthly removal of the net might be a major drawback in the design.

7.6 Conclusion

The information derived from the previous chapters was used to develop five conceptual solutions to prevent the obstruction of the jet pump intakes. The five conceptual solutions were a river abstraction, submerged groyne, pile-and-mesh structure, mobile jet pump, and a coarse material catchnet. All five solutions are considered viable solutions to the current problem at hand, but some were deemed more viable than others for a diversity of reasons.

The conceptual solution that proved the most promising is the pile-and-mesh structure due to the relatively small impact that this proposed solution would have on the surrounding coastline, the low maintenance required, and high capacity of the structure. The second recommended option is the coarse material catchnet, which will also cause a small impact on the surrounding coastline, but will require monthly clearance of the catchnets. The third recommended option is the submerged groyne, which also proved relatively simple in theory, although the major disadvantage of this option is the impact the structure may have on the down-drift shoreline.

The two remaining conceptual solutions are the river abstraction and the mobile jet pump. The major problems with the river abstraction are that the effects of the solution will only be realised in the long term, along with the impact the removal of coarse material will have on the sediment transport. For the mobile jet pump, the costs involved to procure the equipment render this option an unfeasible one. Other than the above-mentioned factors, both these options can still be considered viable solutions.

Chapter 8: Revetment modification and sandtrap clearance

8.1 Introduction

In the previous section, it was mentioned that several of the conceptual solutions required adaptations to the rock revetment and sandtrap. This section addresses the modifications needed in the vicinity of the sand-bypassing system.

The most important factor to address is the presence of coarse material located in the sandtrap. Whether it was supplied by longshore transport, the rock revetment, or remnants from the temporary works is irrelevant. Coarse material must be completely removed in order for the majority of the conceptual solutions explored in the preceding section to perform to their full potential. The continual, periodic removal of coarse material, which is the current method, can still be continued, but this will have no effect on the amount of downtime the Port is experiencing during the year. It is therefore recommended to dredge the sandtrap beneath the jet pumps to its original dimensions in order to remove all the coarse material.

In Chapter 4 it was stated that several of the units have moved from their original positions, while other units fractured. This resulted in the exposure of the under layer of the rock revetment, which migrated into the sandtrap. It is assumed that the reason why these units moved or fractured (also caused by movement) was because certain units in the rock revetment were too small in size to be able to withstand the approaching waves.

The section with the highest percent erosion of the original volume (which is the average eroded area from profile divided by the area of the average original profile x 100), is the section where the most damage occurred (Hudson, 1958). This section is assumed to be located where the core of the revetment was exposed (majority of the armour layer eroded); resulting in the highest damage percentage. The location where the core of the revetment is exposed is shown in Figure 71 below, while the condition of the revetment at this location is depicted in Figure 72 below. The original design of the rock revetment for the section mentioned above according to PRDW (2008) is shown in Figure 73 below.

Figure 71: Location of exposed core.

Figure 72: State of rock revetment.

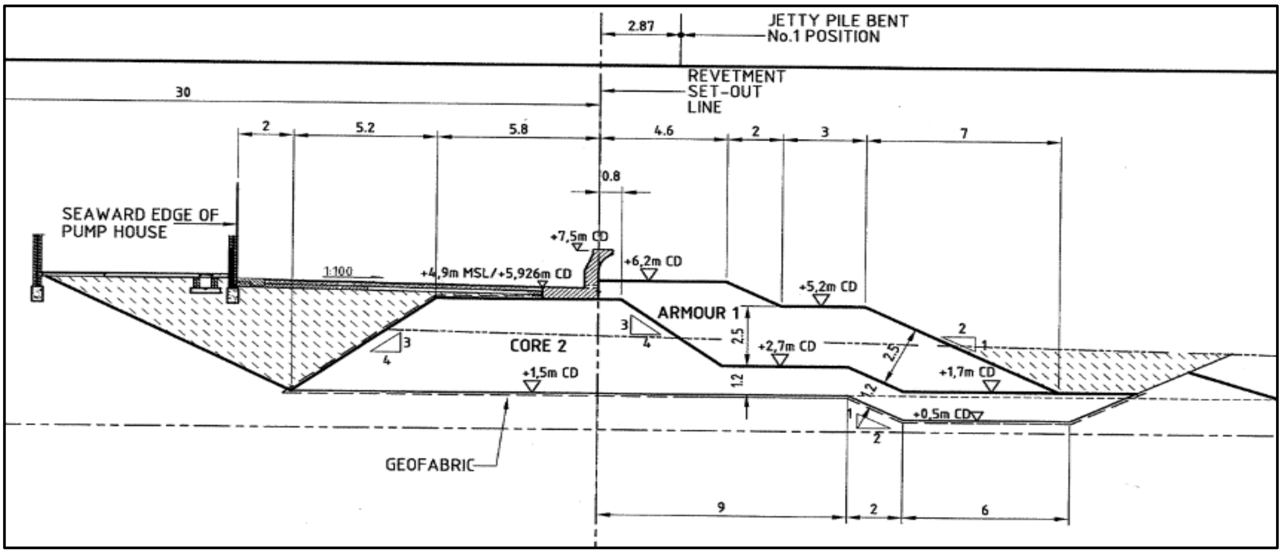


Figure 73: Design of the current rock revetment (PRDW, 2008).

The armour layer and under layer unit weight distribution according to the design in Figure 73 is displayed in Table 12 below. Table 12 indicates that the weight of the armour units vary between 2000 and 9000 kg, while the under layer units vary between 2 and 750 kg.

Table 12: Armour layer and under layer weight distribution (PRDW, 2008).

Armour layer units		**Under layer units**	
Weight (kg)	**% by weight smaller**	**Weight (kg)**	**% by weight smaller**
2 000	< 2%	2	< 2%
3 000	0-10%	5	0-10%
6 000	70-100%	500	70-100%
9 000	>97%	750	>97%

8.2 Replacement of armour units

The simplest approach to solving the problem is to redesign the revetment behind the sand-bypassing jetty and to compare the results with the current revetment, which will reveal whether the armour unit sizes in the current revetment are satisfactory. If not, these units must be replaced with the redesigned armour unit size.

The layout and location of the rock revetment will remain the same, as the only current problem with the revetment is the movement of the armour and under layer units. The only aspect of the revetment that therefore needs to be modified is the armour and under layer units.

The same method will be used to redesign the revetment as for the submerged groyne in Section 7.2. This means that the water depth at the location of the revetment is required. A depth-limited sea state is once again assumed, together with wave breaking conditions at the revetment.

8.2.1 Design water depth

In order to determine the water depth at the location of the revetment, the minimum and maximum water level is required. The difference between these two water levels will be the design water depth at the revetment.

The assumption was made that only the armour and under layer units will be redesigned. This means that the location and layout of the revetment will remain the same. Through this statement it can be assumed that the minimum water level for the new revetment will be at the toe of the current revetment. In Figure 73 the base of the toe is located at + 0.5 m to CD. This results in a minimum water level at -0.52 m to MSL.

The revetment will be designed for breaking wave conditions, which means that wave run-up and wave setup can be excluded from the design water level. The total water fluctuation is equal to the astronomical tide plus the storm surge and SLR, with the results for the different return periods previously shown in Table 10. The conventional design for a coastal structure follows a 1: 100 year return period. As the rock revetment will have an impact on human safety and economic factors, a 1:100 year return period will therefore be used (Sorensen 2005), which is equal to 2.8 m to MSL (Table 10). The design water depth at the revetment is equal to the difference between the water fluctuation and the location of the base of the toe. This results in a design water depth of 3.3 m.

8.2.2 Significant wave height

The relationship between the breaking wave height and the breaking water depth was given in Equation 13 in Section 7.2. The significant wave height for different return periods is shown Table 13 below, using Equation 13, the design water depths from Table 10 and the minimum water level as indicated above.

Table 13: Significant wave height at the toe of the revetment.

Return period in years	**1**	**5**	**10**	**25**	**30**	**40**	**50**	**100**
Significant wave height (m)	1.4	1.6	1.7	1.9	1.9	2.0	2.1	2.6

The significant wave heights from Table 13 will be used to determine the armour and under layer unit sizes of the new revetment.

8.2.3 Armour layer

The armour layer diameter will be determined with Equation 20 below, by using the significant wave height from Table 13 above.

Equation 20: Hudson (1974)

$$M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot\alpha}$$

- M_{50}= median mass of armour layer (kg)
- ρ_s= mass density of armour unit (kg/m^3)
- ρ_w= mass density of water (kg/m^3)
- H = the significant wave height (m)
- K_D = stability coefficient
- α = slope angle (°)

The assumptions made in order to determine the armour layer weight are shown in Appendix A. The required median mass of the armour layer according to Hudson (1974) was determined for the different return periods as shown in Table 14 below.

Table 14: Median mass required for different return periods.

Return period in years	**1**	**5**	**10**	**25**	**30**	**40**	**50**	**100**
Median mass required (kg)	275	420	475	622	677	778	899	1646

It was previously indicated in Table 12 that only 2% of amour units have a mass lower than 2000 kg. Comparing Table 14 with this result indicates that the only storm event that might have an impact on the revetment is a storm with a 1:100 year return period. This is a minor probability because it will only affect 2% of the revetment. This means that the original design of the revetment is able to withstand these large wave events, which indicates that the problem with the armour layer and under layer sizes is not with the original design. Therefore, an alternative factor must be impacting the state of the rock revetment.

8.2.3.4 Modifications

In the previous section it was determined that the original design is sufficient to withstand the wave conditions at the rock revetment. This means that there must be another reason why the armour units are displacing or fracturing.

The disadvantage of using natural rock for the armour layer is that there is a large variation in the quality of the units, which can result in a complete failure, by breakage, of some of the armour units (Jordaan & Bell, 2009). Unfortunately, this process cannot be prevented because the type of armour units was already chosen and the revetment constructed. The only method to decrease this source, is by removal and replacement of the fractured armour units in the revetment, which must be done through continual visual assessment of the revetment state. The other reason why some of the armour units might be displacing or cracking, could be because some of the units were not the correct size as specified by the original design. The only method to solve inadequate sizes is through visual observation of the rock revetment, by selecting the units that might appear to be too small in size. These units that appear to be underweight must be removed and replaced by the correct armour unit weight.

For this study, these two aspects cannot be proven as the reasons for the migration of the armour and under layer units into the sandtrap. Therefore, it is recommended that the area with the most damage, as identified earlier, must be completely reconstructed. The entire armour layer at this section must be removed and replaced to ensure that the correct sizes are used.

8.3 Colcrete Mortar

8.3.1 Introduction

The previous solution was the most basic approach to the revetment problem. An additional solution is by making use of a Colcrete Mortar, which is a product created by Colcrete-von Essen. Colcrete-von Essen is a German based company who specialises in hydraulic engineering projects around Europe.

Colcrete Mortar is a highly durable grouting compound made from cement, water and aggregate, produced in a completely mechanical process which contains no added chemicals (Colcrete-von Essen, 2015). The product is thoroughly mixed in a special Colcrete mixer which creates the colloidal properties of the product.

After the product is prepared, there is no further interaction with the surrounding water molecules, which prevents segregation under water and provides excellent adhesive properties. As soon as the Colcrete Mortar sets, it becomes a waterproof compound (Colcrete-von Essen, 2015).

8.3.2 Application

The Colcrete Mortar is applied to an existing rock revetment as shown in Figure 74 below. The product moves in between the spaces of the rock revetment where it ultimately sets, binding the units together. This creates one large unit instead of smaller individual units. The improved structure can withstand larger waves and currents and also prevent the displacement of armour units.

Figure 74: Application of Colcrete Mortar (Colcrete-von Essen, 2015).

8.3.3 Challenges

The product creates a single unit, which can pose problems as it creates a structure that also moves together. If there is a weak section in the revetment, this section will cause the surrounding sections to move as well, which will affect the entire revetment. Therefore, it will be critical to repair the sections in the revetment where the under layer is already exposed before the application of the Colcrete Mortar.

Another challenging aspect is that Colcrete-von Essen is based in Germany, and is currently the sole company applying this specific product, which limits the information available regarding the product. Therefore, no further design aspects could be determined for this specific situation due to the lack of response on emails from the company.

8.4 Conclusion

In this section, two methods were suggested to help resolve the problems with the revetment. It was determined that the original design of the rock revetment proved sufficient for storm wave conditions. Alternative reasons were stated that might be the reason causing the amour units to move or fracture, but these reasons could not be proven. It was recommended that a specific section in the rock revetment should be reconstructed and that armour units that appear to be too small in size should also be replaced.

Because it was shown that the original design is sufficient, the Colcrete Mortar option could be the ideal solution to the problem, but due to the absence of a response from Colcrete-von Essen, the information available about Colcrete Mortar is very limited. Therefore, this solution can unfortunately not be recommended in this study.

If the recommendations above do not show any improvement, it is recommended that the pile-and-mesh structure or the coarse material catchnet should be expanded around the entire sandtrap as shown in Section 7.2 and 7.5. This expansion will prevent the coarse material from migrating into the sandtrap, but clearance will still be required as stated in each section.

Chapter 9: Conclusions and recommendations

9.1 Conclusion

The construction of the Port of Ngqura was approved on the condition that the Port would make due provision to mitigate the obstruction of the longshore transport caused by the breakwaters. The Port consequently installed a fixed sand-bypassing system with the capability to match the annual longshore transport, which was estimated at 200 000 m^3.

The Port however experienced numerous problems with the fixed sand-bypassing system that prevented the system from reaching the design bypassing rate. The primary problem was the presence of coarse material in the sandtrap. The coarse material builds up, in crater form, around the jet pump intakes; restricting the flow of sand towards the intake and thus preventing the fluidisation of sand and pumping thereof.

While the Port is currently making use of alternative methods to achieve the design rate of the bypassing system, these methods cannot be regarded as permanent solutions. For the bypassing system to function according to the design specifications, the supply of coarse material to the sandtrap must be mitigated. Two alternative fixed sand-bypassing systems, the Nerang river and Tweed river bypassing schemes, were investigated and it was determined that similar problems with debris in the sandtrap arose. The methods that were used by the Nerang was a costly procedure, which cannot be seen as a permanent solution for the Port of Ngqura. Therefore, alternative methods were investigated.

Investigations of the sand-bypassing system revealed that the jet pumps could handle particles of up to 150 mm, which meant that all the coarse material with a diameter larger than 150 mm would cause an obstruction at the intakes. It was therefore necessary to determine the sources of these larger particles, and it was found that the sandtrap was supplied with coarse material from natural sources, the rock revetment, and remnants of the temporary construction works.

The origin and properties of the coarse material supplied by the rock revetment and the temporary construction works were known, which meant that enough information was obtained in order to create a solution for both these sources, which is explained later. The information about the natural source of coarse material was more limited, which led to further investigation regarding the principles of sediment transport and particle motion as well as the origin, volume and properties of this natural source.

Research showed that different particle sizes have different characteristics when it comes to transport thereof. Coarse material (for example cobbles) are more likely to move as bedload transport, whereas smaller particles (for example sand) will more likely move in suspension. The incipient motion of a smaller particle is lower compared to a larger particle, which leads to the assumption that larger particles will most likely be in motion during storm events when the velocities are higher.

The depth of closure for sand and cobbles were calculated to determine up to what depths these two materials will move. The depth of closure for sand was determined as 14 m and the depth of closure for cobbles with a diameter of 150 mm, delivered to a depth of 7 m during storm wave conditions. These results concluded that if a protection measure is used to obstruct the cobble movement in the surfzone, the structure will only need to extend to a depth of 7 m instead of 14 m.

The literature study revealed that the primary source of naturally occurring coarse materials for the area where the Port of Ngqura is located, was the Swartkops river, with an estimated mean annual volume of 150 m^3. The information also revealed that some of the cobbles were derived from local beach rock and coral that are also contained in the system.

The information derived from these investigations was used to develop five conceptual solutions to prevent the obstruction of the jet pump intakes. The five conceptual solutions were a river abstraction, submerged groyne, pile-and-mesh structure, mobile jet pump, and a coarse material catchnet. All five solutions are considered viable solutions to the current problem at hand, but some were deemed more viable than others for a diversity of reasons.

The conceptual solution that proved the most promising is a pile-and-mesh structure due to the relatively small impact that this proposed solution would have on the surrounding coastline, the low maintenance required, and high capacity of the structure. The second recommended option is the coarse material catchnet, which will also cause a small impact on the surrounding coastline, but will require monthly clearance of the catchnets. The third recommended option is the submerged groyne, which also proved relatively simple in theory, although the major disadvantage of this option is the impact the structure may have on the down-drift shoreline.

The two remaining conceptual solutions are the river abstraction and the mobile jet pump. The major problems with the river abstraction are that the effects of the solution will only be realised in the long term, along with the impact the removal of coarse material will have on the sediment transport. For the mobile jet pump, the costs involved to procure the equipment renders this option an unfeasible one. Other than the above-mentioned factors, both these options can still be considered viable solutions.

Some of the conceptual solutions were coupled with revetment and sandtrap modifications. The sandtrap required clearance and dredging to return it to its initial specification. It was concluded that the original design of the rock revetment proved sufficient for storm wave conditions. It was recommended that a specific section in the rock revetment should be reconstructed and that armour units that appear to be too small in size, should also be replaced. If these recommendations do not show any improvement, it is recommended that the pile-and-mesh structure and the coarse material catchnet should be expanded around the entire sandtrap.

The study concludes that each proposed solution together with the required sandtrap and revetment modifications can serve as a potential solution in order to achieve the design bypassing rate of the fixed sand-bypassing system at the Port of Ngqura.

9.2 Recommendations

Based on the research conducted and knowledge obtained through this study, the following recommendations are made:

- The majority of the conceptual solutions were only based on theoretical designs and due to the unconventional methods in which most of the components are used, the performance of these solutions is unknown. Therefore, it is highly recommended that before any of these solutions are implemented, the necessary physical modelling should be executed.

- Throughout this study it was assumed that the volume of coarse material that is transported along the coast is equal to the volume of the coarse material supplied by the Swartkops river. Due to the lack of information available about the volume of coarse material actually originating from local beach rock or reef, this was only a reasonable theoretical assumption for this study. It is therefore recommended that more information must be obtained relating to the origin of coarse material.

- The main principle that the majority of the conceptual solutions were based on, was that if the coarse material in the longshore transport is obstructed, the problem would be resolved. This assumption is true in the long-term, but very little is known regarding the coarse material already located in the vicinity of the sand-bypassing system. Therefore, it is also recommended that a geophysical and bathymetric survey be conducted to determine the position and quantity of coarse material located in the vicinity of the port.

- In the revetment modification chapter, an alternative solution was mentioned, Colcrete Mortar, to prevent the migration of the armour and under layer units. It is highly recommended that this product be further investigated because this can be a very beneficial and simple approach to solving the problem without replacing the majority of the armour units. The lack of response from Colcrete-von Essen restricted this solution, but further efforts could be expended to liaise with this company for more crucial information.

- Further investigation is required regarding the sizes of the amour units in the rock revetment in order to determine if these unit sizes agree with the original design specifications.

- The submerged groyne, pile-and-mesh structure and coarse material catchnet were all designed up to the depth of closure. This was a very conservative approach, however, if more information can be obtained about the coarse material transport rates, these conceptual solutions can drastically be reduced in length, resulting in much lower costs.

References

ABDELHAMID, N.M., 2013. *Design of breakwaters.* Cairo: Cairo University.

ALUMINIUM FEDERATION OF SOUTH AFRICA, 2011. *Corrosion resistance of aluminium.*

ARTEFACTS,2014-last update, Dolos breakwater. Available:
http://www.artefacts.co.za/main/Buildings/bldgframes.php?bldgid=11776 [18/08, 2016].

BAIRD, D., HANEKOM, N.M. and GRINDLEY, J.R., 1986. *Estuaries of the Cape Part II.* 23. CSIR Stellenbosch: CSIR Research Report 422.

BASSON, G.R., 2016. *Abstraction works in rivers.* University of Stellenbosch.

BUREAU OF METEOROLOGY, 2016-last update, Australian Water Information Dictionary [Homepage of Australian Government], [Online]. Available: http://www.bom.gov.au/water/awid/id-535.shtml [22/06, 2016].

CEM, 2006. Fundamentals of design. *Coastal Engineering Manual. 6* edn. Chapter 5.

CERC, 1984. Wave and water level predictions. In: COASTAL ENGINEERING RESEARCH CENTRE, *Shore Protection Manual.* 6 edn. Washington, D.C.: U.S. Army Corps of Engineers, pp. 88.

CHANSON, H., 2004. Sediment transport mechanism. *Hydraulics of Open Channel Flow.* 2 edn. Butterworth-Heinemann, pp. 193.

CIRIA, 2007. *The Rock Manual: The use of rock in hydraulic engineering.* Netherlands: Ciria.

CLAUSNER, J.E., 1989. *Jet Pump Sand Bypassing, Nerang River Entrance, Australia.*

COASTAL AND HYDRAULICS LABORATORY, 2008-last update, Sand Bypassing [Homepage of US Army Corps of Engineers], [Online]. Available: http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=Articles;193&g=42 [15/03, 2016].

COLCRETE-VON ESSEN, 2015-last update, Colcrete Mortar. Available: http://www.colcrete-von-essen.de/index.php/colcrete-mortar.html [19/07, 2016].

COWPER AND THOMAS, 22/01, 2014-last update, The Nerang sand bypass system [Homepage of Pump Industry], [Online]. Available: https://www.pumpindustry.com.au/the-nerang-sands-%E2%80%A8bypass-system/3814/ [15/03, 2016].

CSIR, 1998. *Using the Unibest Model to Investigate Shoreline Evolution at Coega.* Stellenbosch, South Africa: Council for South African Research.

Damen 2016, *Submersible dredge pumps* [Homepage of Damen], [Online]. Available: http://products.damen.com/en/ranges/dop-pump [2016, 01/09].

DAVIDSON-ARNOTT, R.G.D. and GREENWOOD, B., n.d. Waves and sediement transport in the nearshore zone.

DEAN, R.G., 2002. Sand transfer using jet pumps. *Beach Nourishment: Theory and Practice.* World Scientific, pp. 17-19.

DEAN, R.G and DALRYMPLE, R.A., 2004. Coastal Processes with Engineering Applications. *Cambridge University Press.*

DU PLESSIS, G., 2010. *Construction of the Port of Ngqura: An Overview.* Civil Engineering.

DREDGE BROKERS, 2016-last update, Hydralic dredgers for sale [Homepage of Dredge Brokers], [Online]. Available: http://www.dredgebrokers.com/hydraulic-dredgers [16/08, 2016].

GAMELKOORN, S., 15/08/2013, 2013-last update, Port of Ngqura Container Terminal. Available: http://portandcorridor.org/rfq-port-of-ngqura-container-terminal [03/11, 2016].

GODA, Y., 2000. *Random Seas and Design of Maritime Structures. 2* edn. Advanced Series on Ocean Engineering.

GOLD COAST WATERWAYS AUTHORITY, 30/07/2015, 2015-last update, The Sand By-Pass System. Available: http://www.gcwa.qld.gov.au/userfiles/resources/static/GCWA_Sand_Bypass_Fact_Sheet.pdf#page=1 [15/03, 2016].

GOSCHEN, W. and SCHUMANN, E., 2011. The physical oceanographic processes of Algoa Bay, with emphasis on the western coastal region: a synopsis of the main results of physical oceanographic research undertaken in and around Algoa Bay up until 2010. *SAEON and IMT.*

GRAVESEN, H., 2008. *Rubble mound breakwaters.* Technical University of Denmark.

HARRIS, L.R., 2012. *An ecosystem-based spatial conservation plan for the South African sandy beaches.*

HOLMES, P., 2001. Coastal and offshore structures. *Coastal defense systems.1* edn. St. Lucia, West Indies: The University of the West Indies.

HUGHES, S., n.d. *Scour and scour protection*, Coastal and Hydraulic Laboratory US Army Corps of Engineers.

ILLENBERGER, W.K., 2008. *Ngqura harbour beach profiling measurements.* Illenberger & Associates.

ILLENBERGER, W.K., 2008. *Ngqura harbour beach profiling measurements.* Illenberger & Associates.

ILLENBERGER, W.K., 1992. *Sediment dynamics of the Sundays River mouth area, Algoa Bay*, University of Port Elizabeth.

JANSEN, W., 2015. *Port of Ngqura: Sand-bypass action plan.* Transnet.

LUND UNIVERSITY, 2014. *Bed Load and Suspended Load.* Lecture.

NIELSEN, P., 1992. *Coastal bottom boundary layers and sediment transport.* Singapore; River Edge, N.J.: World Scientific.

ONLINECRANES, 2016-last update, Cranes for sale. Available: http://www.onlinecranes.com/index.php?search=OC [18/08, 2016].

PATEL, S.R. and MOES, J., 2002. *Feasibility Study on the Potential of Wave, Current and Tidal Energy along the Coast of South Africa.* Stellenbosch: CSIR.

PIANC, 2015. *Anti-Sedimentation system for marinas and yacht harbours.* International Council of Marine Industry Associations.

PRDW, 2008. *Rock revetment protection to the sand bypass pumping station platform.* Cape Town, South Africa: Prestedge Retief Dresner Wijnberg.

PRDW, 2001. *Sand Bypass: Detailed Design Report.* Wijnberg, Cape Town, South Africa: Prestedge Retief Dresner Wijnberg.

PRDW, 2001. *Summary of Field Data on Coastal Processes.* Cape Town, South Africa: Prestedge Retief Dresner Wijnberg.

PRDW, 2000. *Coastal Processes: Beach Monitoring.* Cape Town, South Africa: Prestedge Retief Dresner Wijnberg.

PRDW, 1999. *Siting of Main Pump Station - Aeolian Sand.* Cape Town, South Africa: Prestedge Retief Dresner Wijnberg.

PRDW, E.A.C., 1997. *Algoa Bay Industrial Port: Feasibility Study.* Cape Town, South Africa: Prestedge Retief Dresner Wijnberg.

RIEDEL, H.P. and BYRNE, A.P., 1986. RANDOM BREAKING WAVES HORIZONTAL SEABED. *Coastal Engineering Proceedings,* **1**(20).

RUTHERFORD, K., 2015. *Shoreline changes and longshore transport at the Port of Ngqura.* Master's in Civil Engineering edn. 2015: University of Stellenbosch.

SCHOONEES, J.S., 2015. Design of Rubble Mound Structures, *Ratio of armour layer to underlayer* 2015, pp. 1.

SCHOONEES, J.S. and THERON, A.K., 2002. Development of an accurate longhsore sediment transport model. Journal of the South African Institution of Civil Engineering, pp. 12-17.

SCHOONEES, J.S. and THERON, A.K., 1993. *Beginning of longshore transport for coarse sediment.* Stellenbosch: CSIR.

SCHOONEES, J.S., 2001. *Longshore sediment transport: applied wave power approach, field data analysis and evaluation of formulae*.

SORENSEN, R.M., 2005. Return period of waves. *Basic coastal engineering.* 3 edn. Springer Science & Business Media, pp. 188-189.

SWART, H.E., 1986. *Physical environmental interactions in the Sundays River/Schelmhoek area.* Stellenbosch, South Africa: Council for Scientific and Industrial Research.

TERRAM, 2015-last update, Geotextile filters in marine breakwaters. Available: http://www.terram.com/market-sector/coastal-and-waterways/?aaopen=geotextile-filters-in-marine-breakwaters [30/06, 2016].

THE OPEN UNIVERSITY, 1999. Chapter 4 - Principles and Processes of Sediment Transport. In: T.O. UNIVERSITY, ed, *Waves, Tides and Shallow-Water Processes.* Oxford: Butterworth-Heinemann, pp. 96-124.

THERON, A.K., 2014. *Scoping and EIA for the Provision of Marine Infrastructure, including a General Cargo Berth and Liquid Bulk Berths at the Port of Ngqura, Nelson Mandela Bay Municipality. Chapter 8: Shoreline Impact Assessment.* Transnet.

THERON, A.K., 2016. *Methods for Determination of Coastal Development Setback Lines in South Africa*, University of Stellenbosch.

THERON, A.K. and SCHOONEES, J.S., 1994. *Port of East-London: Planning and influence of breakwater.* Pretoria: WNNR.

TINLEY, K.L., 1985. *Coastal dunes of South Africa.* National Scientific Programmes Unit: CSIR.

TRANSNET, 2016. *SAND BYPASS OPCO PRESENTATION.*

TRANSNET, 2010-last update, Port of Ngqura. Available: http://www.transnetnationalportsauthority.net/OurPorts/Ngqura/Pages/Overview.aspx [16/03, 2016].

TRANSNET NATIONAL PORTS AUTHORITY, 2013. *Port of Ngqura- sand bypass.* Transnet.

TWEED SAND BYPASS, 20/01/2016, 2016-last update, The permanent sand bypassing system. Available: http://www.tweedsandbypass.nsw.gov.au/operations/general-information [14/03/2016, 2016].

TWEED SAND BYPASS, 02/2016, 2016-last update, Sand Pumping Details. Available: http://www.tweedsandbypass.nsw.gov.au/__data/assets/pdf_file/0017/521117/pumping-table-2016-02.pdf [14/03/2016, 2016].

U.S. ARMY CORPS OF ENGINEERS, 2012. TYPES AND FUNCTIONS OF COASTAL STRUCTURES. *Coastal Engineering Manual.* pp. 2-3.

VAN RIJN, L.C., 2007. Unified view of sediment transport by currents and waves. I: Initiation of motion, bed roughness, and bed-load transport. *Journal of Hydraulic Engineering,* **133**(6), pp. 649-667.

VAN DER MEER, JENTSJE W and DAEMEN, I.F., 1994. Stability and wave transmission at low-crested rubble mound structures. *Journal of waterway, port, coastal, and ocean engineering,* 120(1), pp. 1-19.

VAN NIEKERK, A., 2011. *Elevation data for setback lines (LIDAR).* SETBACKS 2 Seminar & Workshop: Development of coastal setback lines edn. University of Stellenbosch.

VERHAGEN, H., 2014. Rock structures and probabilistic aspects, 24/02/2010 2010, TU Delft.

WEEKEND POST, 2013. Harbours playing vital role.

WIJNBERG, A.R., 1993. *Design sea levels for Southern Africa*, University of Cape Town.

WOOLRIDGE, T.H., KLAGES, N.T. and SMALE, M.J., 1997. *Specialist report on the nearshore environment.*

WRIGHT, L. and SHORT, A.D., 1984. Morphodynamic variability of surf zones and beaches: a synthesis. *Marine Geology,* 56(1), pp. 93-118.

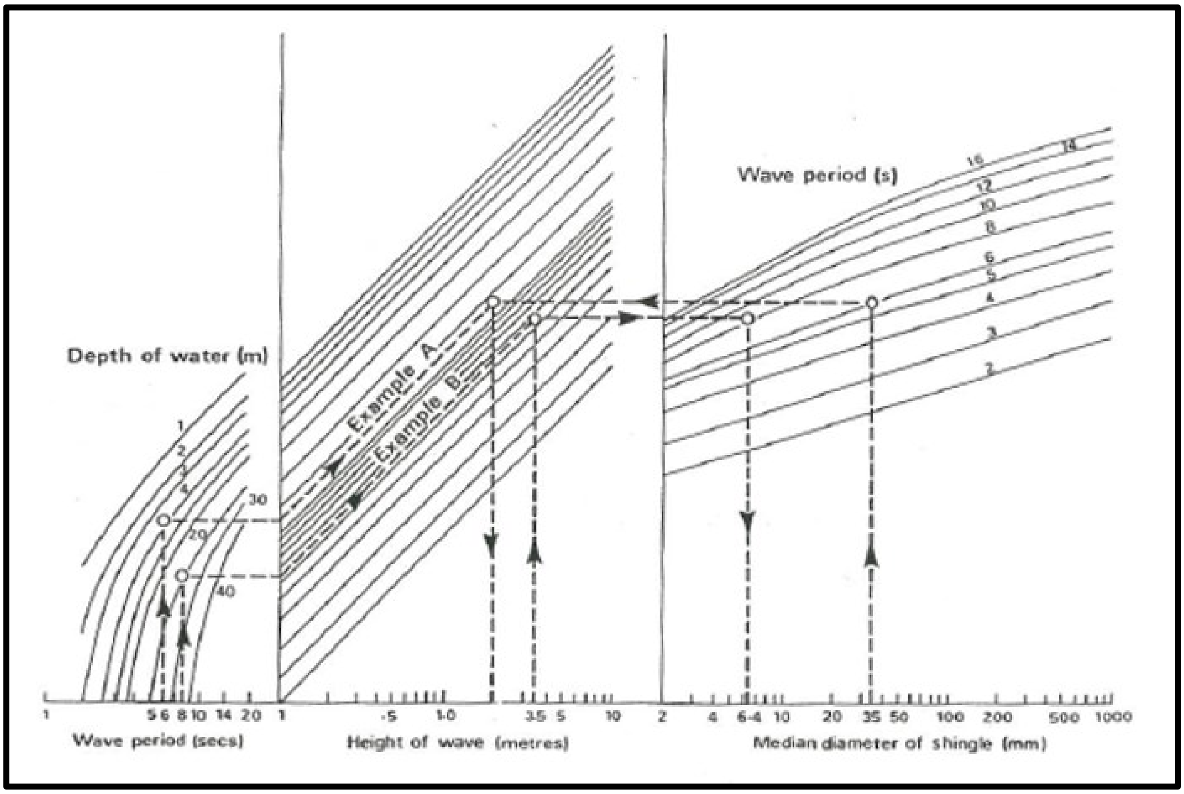
Appendix A

Depth of closure

The assumptions that were made in order to determine the depth of closure are as follows:

- in order to convert D_{50} (grain size exceeding 50% of the sample mass) to D_{90} (grain size exceeding 90% of the sample mass) the method of Schoonees (2001) were used as follows: $(D_{90} = (1.656 \times 10^6 D_{50} - 65.143)/10^6$;
- McCowan (1891) theoretically determined the breaker index as $\gamma = 0.78$ for a wave travelling over a horizontal bottom. This is commonly used in engineering practice as a first estimate of the breaking wave height at a particular depth (USACE,2002b)
- The value of D_{50} is determined by sieving a sample, where D_{50} is the median diameter. Because for larger stones sieving is impossible the value of D_{50} is replaced by D_{n50}, which is often called the median nominal diameter. The relationship according to Verhagen (2014) is as follows $\frac{\mathrm{Dn}_{50}}{\mathrm{D}_{50}} = 0.84$;
- the gravitation acceleration = 9.81 (m/s^2) (CEM, 2006);
- group velocity in shallow water $\mathrm{nC} = \sqrt{\mathrm{gd}}$ (CEM, 2006);
- density of seawater $\rho_\mathrm{w} = 1025\mathrm{kg/m}^3$ (CIRIA, 2007);
- density of quartzite which is the assumed sediment $= 2650\ kg/m^3$ (CIRIA, 2007); and
- $K_\Delta =$ layer coefficient $= 1$ for smooth and rough quarry stone (CEM, 2006).

Hydraulics Research method (n.d)



Groyne design

The following assumptions were made regarding the design of groyne:

- The density of seawater $\rho_w = 1025 \mathrm{kg/m^3}$ (CIRIA, 2007);
- density of quartzite, which is the assumed sediment $= 2650\ kg/m^3$ (CIRIA, 2007);
- the gravitation acceleration = 9.81 (m/s^2) (CEM, 2006);
- the slope of the groyne is assumed to be 1:2, resulting in an initial damage level (S) = 2 (CEM, 2006);
- $N_{od} = 0.5$ for the start of damage (CEM, 2006); and
- the crest height is taken as two armour unit diameters.

Revetment design

The following assumptions are made regarding the design of the rock revetment:

- density of seawater $\rho_w = 1025 \mathrm{kg/m^3}$ (CIRIA, 2007);
- density of quartzite which is the assumed sediment $= 2650\ kg/m^3$ (CIRIA, 2007);
- the armour layer units that will be used is rough angular with random placement and a damage factor of 0-5% (CEM, 2006);
- the significant wave height was determined for breaking conditions;
- the slope angle of 2 is assumed which is similar to the original design; and
- K_D = stability coefficient = 3.5 (SPM,1977)